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New Bridge of the Kanawha & Michigan Ry. Over the Ohio River at Pt. Pleasant, W. Va.

By J. A. STOCKER.*

The Kanawha & Michigan Railway crosses the Ohio River at Point Pleasant, W. Va., on a single-track bridge three-quarters of a mile long with a vertical clearance over the channel of 90 ft. above low water. This bridge was rebuilt during the past three years. The new structure is shown by the view Fig. 1 and the diagram elevation Fig. 2.

The original bridge, built in 1882-4, consisted of a steel viaduct approach on the north end 930 ft. long, five Whipple truss spans over the river (two of them appear in Fig 5), and a steel viaduct approach on the south end 1,516 ft. long. The greater portion of the latter was on a 5° curve. The channel span was level, the four other river spans were on a 0.5% grade and both viaducts had 1½% grades reduced to 1¼% on the 5° curve. These grades have always required pusher service, three engines being required to handle a train which one engine was able to handle elsewhere on the division. Operation in this manner was necessarily expensive and traffic was usually congested at this point. When it became necessary therefore to rebuild the bridge on account of heavier train loads, it was decided to relocate the line for three miles south of the bridge and obtain the standard gradient of the Kanawha & Michigan Ry. against northbound traffic, 0.3% compensated for curvature.

The new bridge and south approach have been rebuilt on a 0.2% grade. The north approach has been rebuilt on the original 1½% grade, but both masonry and steelwork are so designed and built that the viaduct may at any time be easily raised to the standard grade against southbound traffic, namely, 0.5%. Span No. 1, the northerly span of the river bridge, has been built on the standard 0.5% grade; span No. 2 has been rebuilt level; spans 3, 4 and 5 were first rebuilt on the original 0.5% grade and then were jacked up to the final 0.2% grade when the south viaduct was rebuilt, a year afterward. The south viaduct and the revised approach location and grading were completed two months ago.

*Assistant Engineer of Construction, Hocking Valley Ry., Columbus, Ohio.

River Bridge.

The new river crossing, which uses the old piers, consists of five spans: four Pratt trusses with inclined top chord and one 415½ ft. Baltimore truss (see also Fig. 2). They were designed for a live-load of 5,000 lbs. per lin. ft. plus a concentration of 50,000 lbs., while the approach viaducts were designed for 5,000 and 60,000 lbs. The old structure which the new bridge replaces had been designed for two 81-ton consolidation locomotives followed by 2,240 lbs. per lin. ft.

The channel span (No. 2) was rebuilt in the summer of 1907. The other spans were rebuilt (in the order Nos. 3, 4, 5, 1) in 1908. All of the rebuilding was done on timber falsework resting on piles. The falsework was built, the floor of the bridge wedged up on it, the old trusses taken down, the floor system exchanged panel by panel, and finally the new trusses erected and swung. The traveler is sketched by Fig. 3. It handled its material from cars on the bridge track.

The falsework comprised about 800 wooden piles about 40 ft. long, and 700,000 ft. B. M. sawed timber; however by re-using the falsework it did duty for two to three times that amount.

The old bridge with its shallow shoes and suspended floorbeams required the masonry to be near the track level. While the new bridge affords the same clearance over the river that the old one had, it is so designed that it was necessary to cut down the piers by about 8 ft. In order to jack the new spans up to final grade, the shoes were made deeper than usual and arrangements made to bolt frames to them for the jacks to lift on. I-beam grillages full length of the pier were placed under each pair of shoes, to strengthen the piers and to afford footing for the jacks (Fig. 4).

Grillages were not placed under the channel span when it was erected, but when the other spans were erected it seemed advisable to strengthen the channel span piers as much as possible; hence special grillages were built for those piers, the channel span shoes were undermined 15 ins. and the grillages placed under the shoes (Fig. 4) with a stone cap between the grillages and shoes. The other piers have no stone caps, and the bridge shoe grillages bear directly on the pier grillages.

When the spans were jacked to final grade, the additional height was made up by steel grillages placed crosswise on the pier grillages thus tying the pier together both longitudinally and transversely. The grillages were then concreted in and a concrete coping formed immediately beneath the bridge seats. Spans 3, 4 and 5 had to be raised by amounts varying from 8½ ins. to 2 ft. 2⁹/₁₆ ins., to conform with the final grade.

One-half mile north of the bridge the Kanawha & Michigan Ry. connects with the Hocking Valley Ry. by a Y. Between the legs of the Y is a plot of ground that was furnished with spur tracks and utilized for a material yard. All falsework and bridge material was handled from this yard. At times there were two work trains, one handling falsework material, the other handling bridge steel, moving back and forth between the material yard and the bridge on the main track, besides the regular traffic amounting to about 30 train movements during daylight hours. It required considerable thoughtfulness on the part of all concerned to avoid serious entanglements and delays. Telephone connections between the yard at Kanauga at the Hocking Valley connection, the bridge, Point Pleasant at the south end of the bridge, and Wagner yard a mile south of the bridge, furnished a ready means of communication between all the busy points. As there were telegraph

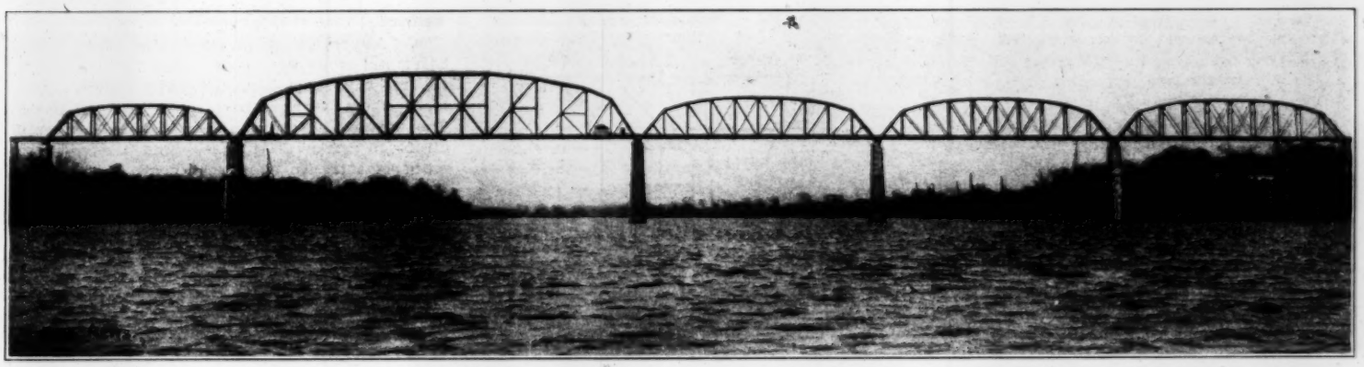

FIG. 1. GENERAL VIEW OF NEW BRIDGE OVER OHIO RIVER AT POINT PLEASANT, W. VA.; KANAWHA & MICHIGAN RY.

operators at Wagner, Point Pleasant and Kanauga, as well as a special operator out on the bridge, approaching trains were anticipated and track cleared for them when they came in sight, thus avoiding stopping trains on the heavy approach grade.

For the erection of the channel span the Government granted permission to block the channel for three months beginning July 1, 1907. On account of continued high water that summer, an extension of time was granted amounting to twenty-five days. Two drop-hammer pile drivers mounted on barges drove the falsework piles. Most of the piles were driven to refusal in 8 to 12 ft. of gravel. The framing of the timber bents followed close after the pile driver. The timber was delivered on flat cars on the bridge above and lowered to the work by two hoist engines placed on the lower chords of the old trusses. The number of bridge carpenters employed varied from 40 to 60. The railway company installed an air compressor at the north end and laid a 2-in. main to the work. Pneumatic tools were used to bore the bolt holes, and also to drill holes in the stone piers for wedging off their top sections.

While the falsework was being built by the railway company, the bridge company was framing the traveler and unloading the structural steel in the material yard. When the falsework was ready for service, the bridge company erected a stiff-leg derrick on the upper chords of the south end of old span No. 1 and by that means unloaded the traveler sections from cars and erected the traveler in place. After the traveler was erected, the old span was wedged up on the falsework and the old trusses taken apart, removed and loaded on cars for shipment. The settlement of the falsework was only a fraction of an inch when the full load came on.

Thus far in the operation of rebuilding there was no interference with the running of trains over the bridge at the will of the dispatchers. The removal of the old floor system and the placing of the new one was a different matter, however. By arrangement with the dispatchers, extra trains were handled in such a manner that the bridge company could be given the right of track between certain scheduled trains each day without interfering with extras. Thus the bridge company was able to work without interruption from 8.15 a. m. until 1 p. m. and from 2 p. m. until 4.30 p. m.

The old trusses were removed between Sept. 9 and 16, 1907, the floor systems were exchanged between Sept. 16 and 23, and the span was put in service Oct. 8.

The deck of the falsework was hoisted and loaded on cars on the track above. The timber of the bents was unbolted one deck of each bent at a time and thrown over into the river where it was caught and rafted. After all the timber was rafted, the rafts were towed about a mile up the Kanawha River to a tie hoist operated by the railway company. There it was hoisted from the river, loaded on cars, and returned to the material yard and stacked, as it was to be used in the work of rebuilding the remaining spans the following year. A dredging company was employed by the day to remove the piles. The 315 piles were removed and those that were worth saving loaded on barges and delivered at the hoist. This work was done for $1,167.

The timber was framed and erected at a cost of $7.65 per M. ft., exclusive of work train service and machinery rental. The average wages paid the carpenters was 22½ cts. per hr.

In rebuilding the other four spans, in 1908, the work was handled in the same manner, except that as soon as one span of falsework was released from service it was torn down, moved ahead to another span and rebuilt. In all cases the deck of the falsework was hoisted and handled from the bridge above while the bents were unbolted and the timber thrown into the water and floated ahead to the new position. When the last span was completed, that part of the falsework that was over the water was thrown down, rafted and hoisted at the tie hoist. The part that was over dry land was hoisted and loaded on cars on the track above. The piles were pulled by contract, 240 at $2.50 apiece loaded on barges furnished by the railway company, and 62 at $3 apiece, these being more difficult to get at.

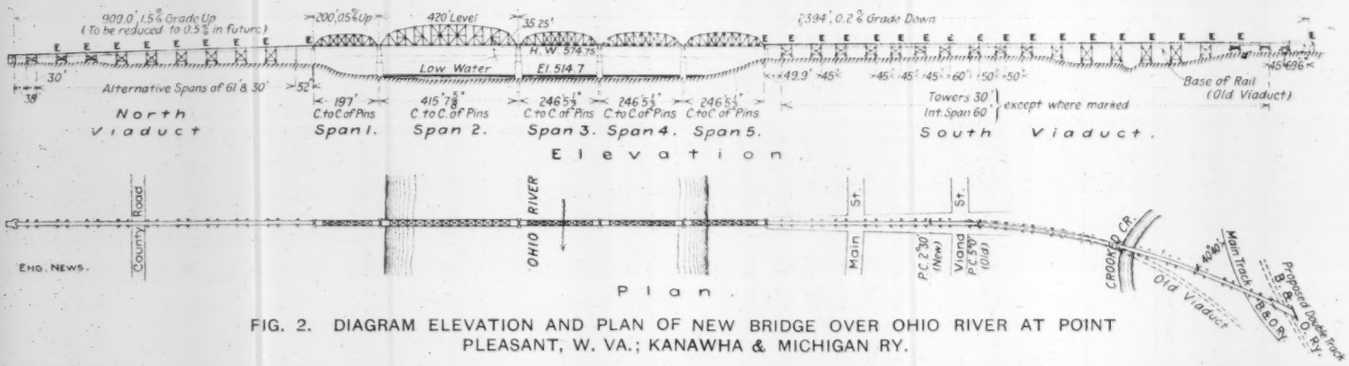

FIG. 2. DIAGRAM ELEVATION AND PLAN OF NEW BRIDGE OVER OHIO RIVER AT POINT PLEASANT, W. VA.; KANAWHA & MICHIGAN RY.

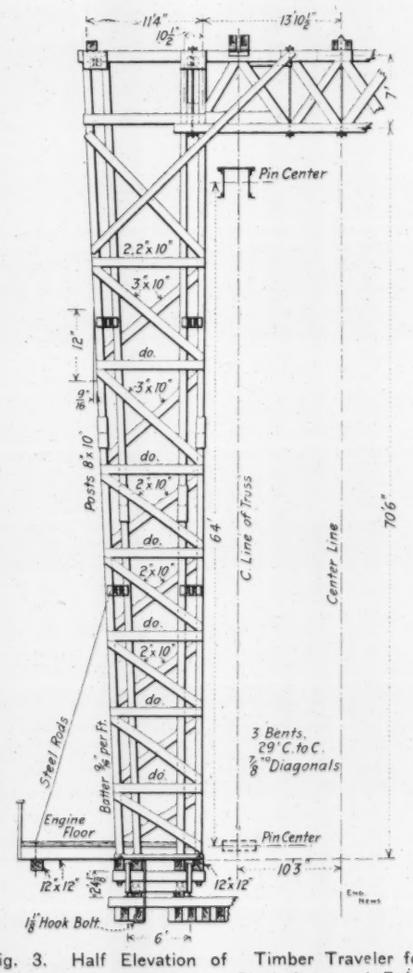

Fig. 3. Half Elevation of Timber Traveler for Removing and Erecting River Spans of Point Pleasant Bridge.

North Approach Viaduct.

The north approach viaduct (see sketch Fig. 7 and views Figs. 8 and 9) consists of deck plate-girder spans supported by towers. The tower spans are 30 ft. long and the intermediate spans are generally 61 ft. long. All the girders are 8 ft. ¼-in deep back to back of angles and are spaced 8 ft. on centers. They were shipped to the job in pairs with all crossbracing in and completely riveted. The towers consist of four posts built up of Z-bars and plates and braced throughout with stiff members made up of latticed angles and channels.

The towers stand on reinforced-concrete pedestals with sawed sandstone caps. The pedestals were molded in forms made in two sections each 4 ft. high, consisting of four separate sides held in place by means of wooden wedges in slots in the interlocked side strips. Eight of these forms were used and they gave very satisfactory service. All reinforcement is twisted steel. The horizontal loops were bent to true dimensions in the railway shops and the ends welded. This made them very convenient to put in place. The surface of the concrete was rubbed until all form marks disappeared, and then washed with cement. No effort was made to finish the pedestals at the exact level, but their tops were trimmed to grade after the concrete had set, and the stone caps were set in a mortar bed, using an engineer's level to insure proper grade.

In the case of the north viaduct, the pedestals were finished at such a size on top that when the viaduct is raised to final grade, they may be built up and finished standard size at final grade. In order to accomplish this, the forms were set on the footing with the top of forms at proper height for final grade. The forms were then filled only to the height necessary for the present grade. Hence, when the viaduct is raised to the final grade, the same forms may be put in place again and filled to the top.

In anticipation of raising the viaduct to the 0.5% grade, it was so designed that the posts do not stand plumb but the tops incline toward the north about 17½', half the angle between a 0.5% and a 1½% grade. The posts are finished square at the foot and rest on bearing plates whose faces are also inclined to each other 17½'. When the viaduct is raised to the final grade, the posts will incline the same amount in the other direction, and the bearing plates under the posts will be reversed.

The north viaduct was rebuilt during the winter of 1908. In designing, the span lengths had been chosen such that the new bents stand about midway between the old ones. The old girders were 3 ft. deep and 9 ft. c. to c., and the batter of the posts was the same as that of the new posts, 2 ins. per ft. It was possible, therefore, to erect all the new posts and bolt up the greater part of the tower bracing before disturbing the old structure. The new towers were built in place piece by piece by means of a derrick car running on the track above. After the towers were erected as complete as possible without tearing down the old structure, the work of placing the girders began.

The erectors were given the right of track between scheduled trains from 8.15 a. m. to 1 p. m. whenever they requested it. They usually put in one 30-ft. and one 61-ft. span during each working period. The work of tearing down the old structure was handled with one derrick car, while the new girders were handled with another. The girder spans, fully cross-braced and riveted,

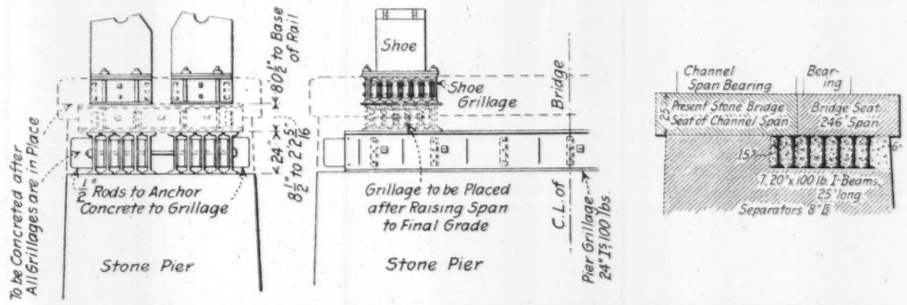

FIG. 4. STEEL DISTRIBUTING GRILLAGES UNDER NEW SHOES, POINT PLEASANT BRIDGE.
(Sketch at right shows grillage placed on channel-span piers after rebuilding of channel span.)

were picked up in the material yard, placed on a low truck car, and derrick car girder and all pushed up the main track nearly to the point where the span was to be placed. Then the derrick picked up the girder and car together, and with it suspended in the air was moved forward until the girder hung directly over the posts on which it was to be placed. Now the girder was carefully lowered on to the posts and bolted into position. The derrick then took a hitch on the truck car and swung it out from beneath. The new ties, previously framed and distributed under the viaduct, were hoisted and put in place, and the rails spiked down. The temporary connections between the new and old work were made by means of timber stringers.

The new 61-ft. spans weigh 60,500 lbs. each.

South Viaduct.

The replacement of the South viaduct was the last piece of work undertaken* because the interference of old and new viaducts made it necessary to await the relocation and grading of the approach. The work was done in the spring of 1909, being the third year of work on this bridge.

The original viaduct had 600 ft. of tangent on a 1½% grade and 916 ft. of 5° curve on a 1¼% grade. Spans 3, 4 and 5 of the river bridge, originally 0.5% grade and to be changed to 0.2%, were rebuilt on their original grade, as already stated, and jacked up to final grade later when the South viaduct was erected.

As the revised alinement substitutes a 2° 30' curve for the 5° curve in the old viaduct, and the P. C. is 143 ft. farther north, the new center line swings to the west of the old, then crosses at a very sharp angle and swings to the east. At the intersection of the old and new center lines the grades differ by 14 ft. The rebuilding of this viaduct under traffic was therefore a difficult matter. Two schemes were proposed. The first involved lining the viaduct to the west sufficiently to clear the new one, except at the north end, where a reverse curve would be needed to reach the river bridge, which would interfere somewhat with the new work. This scheme would necessitate handling all the traffic over the old viaduct on temporary foundations, with excessive curvature and a 1½% grade, a difficult matter. The other scheme proposed was to build the South viaduct from the south end to the point of interference with the old one, then throw down enough of the old one to allow the erection of six spans of the new work and at the same time line the remainder of the old viaduct to the line of the new and jack it up to the new grade. The latter scheme was adopted. It was carried out as follows:

OPERATION 1.—This comprised all the work that could be done without disturbing the old viaduct, and also the work of installing the necessary blocking for jacking and shifting portions of the old viaduct in the next operation.

Falsework and blocking was distributed to the old viaduct between Old Bents 6 and 20 (Fig. 11), and needle beams consisting of 12-in. 31½-lb. I-beams were threaded through the posts of each bent in this length to give a hold for the jacks.

At the same time the new viaduct was erected complete from its southerly end to New Bent 26 (between Old 34-35) by derrick-car; this part is marked "Section A" in Fig. 11. Bent 26 could not be fully erected, because its west column fouled the train clearance on the old structure by nearly 2 ft. This column was set in an inclined position, the bracing being left out, and the girder which should rest on it was set off line on blocking, ready to put the bent and span in final condition as soon as the old structure should be abandoned.

The portion just north of this, New Bents 16 to 26, which interfered with the old viaduct (Bents 20-33), was to be erected in one operation after throwing down the old work. As much of the bents was erected as could be without fouling the old work, and the remaining material was placed alongside where it could be picked up and set by a derrick-car running on the viaduct.

That part of the old viaduct marked "Section C" on Fig. 11, bents 16 to 20, was to be skidded over in line with the new structure, to be then replaced span by span with new work. Together with the eleven spans lying just north, it was to be jacked up to a level connecting with the grade of the new structure and marked "Grade A" in Fig. 11. Skidways and blocking were placed for this series of shifts before any part of the old structure was disturbed.

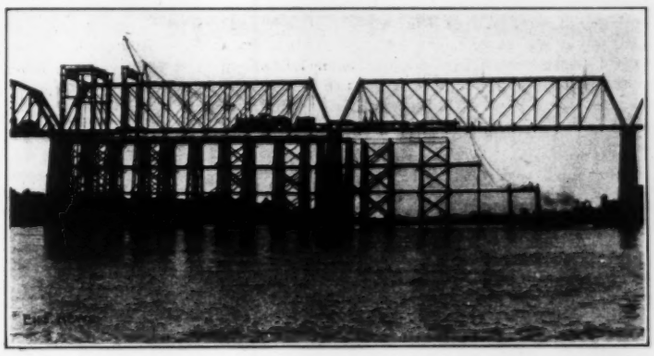

FIG. 5. TWO OF THE OLD WHIPPLE SPANS OF THE POINT PLEASANT BRIDGE.

OPERATION 2.—Making a traffic connection between the south half of the new viaduct and the north half of the old was the second operation. It comprised tearing out the short interference section of the old work ("Section B," Fig. 11, bents 21-34), building the new bents and spans in this section, jacking the northerly part of the old structure (Old Bents 6 to 20) to the proper line and grade, and shifting Bents 16 to 20 to the new alinement.

The railway company stopped operating trains over the old structure at 10 p. m. Sunday, June 20, 1909. The bridge company's forces immediately began work tearing out the old structure where necessary and finishing the erection of the new towers 16 to 26 by derrick car operating on the old structure. The work was lighted by arc lights and lanterns. At 4 a. m., June 21, the placing of the new girder spans in Section B was begun by derrick-car operating on the new structure and a force was set to work jacking the old structure to Grade A and shifting Section C to the new alinement. There were forty of the railroad company's bridge men at the jacks and fifty of the bridge company's men erecting steel. All day long the work continued incessantly until 6.30 p. m., at which time ten spans (430 ft.) of new viaduct had been built, 480 ft. of old viaduct had been jacked up, a temporary connection had been made between the old and new work and the track laid ready for trains.

During the twenty hours that traffic over the viaduct was stopped, the K. & M. passenger trains were run from Point Pleasant to Clifton (10 miles) over the B. & O Ry., and the pass-

*Except jacking the North viaduct to a higher level with reduced grade, as noted. This work is postponed until the approach is relocated.

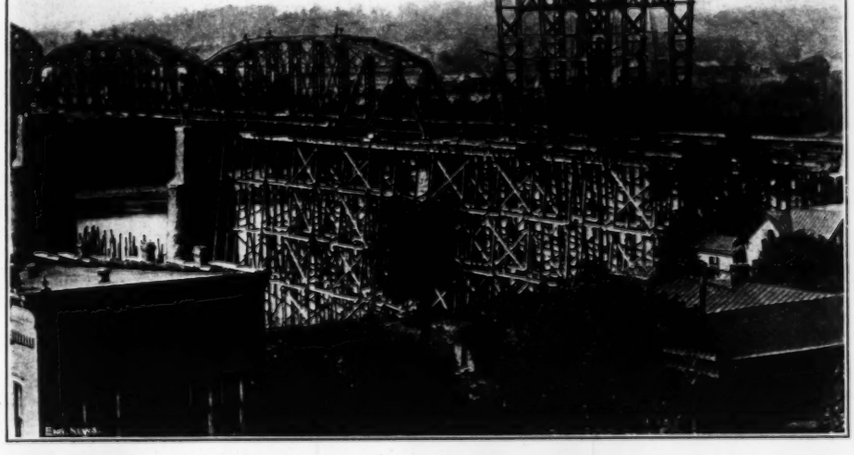

FIG. 6. REBUILDING SPAN 5, POINT PLEASANT BRIDGE.

engers ferried across the Ohio River to the K. & M. Ry. at Middleport, opposite Clifton. Through freight traffic was entirely stopped.

OPERATION 3.—Traffic being restored on the structure, the structure was gradually rebuilt to final grade as far north as New Bent 7, and the old structure north of this brought to Grade B, i. e., connecting the new grade at Old Bent 9 with the old grade of the river bridge.

This involved jacking up the old structure from Bent 20 to Bent 9 to the final grade, and the part between Bent 9 and the river bridge to the 0.6% grade marked Grade B. Then Old Bents 20 to 9 were torn out, one by one, and new bents and spans substituted, in just the same way as employed previously on the North viaduct. Traffic was stopped between 8.15 a. m. and 1 p. m. whenever necessary, to accommodate this reconstruction work.

OPERATION 4.—The viaduct was completed after another change of grade, which necessarily involved the south end of the bridge, as follows: The south end of Span 5 of the bridge was jacked to final grade, and simultaneously the old viaduct from the bridge to Old Bent 8 was jacked up from Grade B to final grade. Then the remainder of the old viaduct was torn out, span by span, and replaced by new structure, from New Bent 6 to the bridge.

OPERATION 5.—Finally, the river bridge was brought up to grade, by jacking up the north end of the shore span (Span 5), both ends of Span 4 and the south end of Span 3. In this operation intermediate grillages were placed under the shoes, between pier grillage and shoe grillage, as sketched in Fig. 4.

Riveting all connections of the viaduct was now done, the structure having been bolted up to this time. Fig. 12 gives a picture of the viaduct at this stage, with the southerly portion of the old structure still standing. The latter was taken down and removed at leisure.

The erection work of this viaduct was done like that of the North viaduct, namely, erecting fully assembled spans by derrick-car, the girders being riveted up complete with their bracing in the shop. The through girder span at the extreme south end, and the deck span over the B. & O. tracks (Bents 34-35), were built in place piece by piece.

DESIGN.—The design of the viaduct was planned to suit the erection. The new bents were placed between the old ones, so that the footings and the lower parts of the towers could be built without removing parts of the old structure.

On the 2½° curve the tower centers are on the center line, and as all towers are square the centers of the individual bents are slightly off the center line. The superelevation, 1¾ ins., is obtained by building the outside post higher than the inside one and tilting the girders toward the inside of curve. This allows of the use of ties of uniform thickness and dapped equally at both bearings. It also causes the girders to stand nearly parallel to the resultant of vertical load and centrifugal force.

A special feature of the south viaduct is the extremely shallow girder span over the Baltimore & Ohio railway tracks made necessary by clearance requirements. The bents under this span do not rest on pedestals but on collision piers

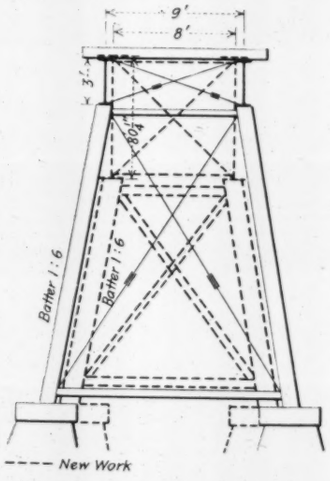

Fig. 7. Sketch Showing Relative Positions of Old and New Posts and Girders, North Approach Viaduct of Point Pleasant Bridge.

standing 6 ft. above the B. & O. tracks, so that in case of derailment of a B. & O. train there will be no probability of knocking these bents down. The skewed bent north of the proposed B. & O. doubletrack is too long for a continuous pier under both legs and therefore two small collision piers are used.

The truss spans were designed, built and erected by the Mount Vernon Bridge Co. The falsework, however, was handled by the railway carpenter force. The two approach viaducts were designed, built and erected by the American Bridge Co.

The entire work was done under the direction of Mr. Wm. Michel, Engineer of Maintenance-of-Way of the Hocking Valley Ry. and the Ohio Central Lines. The writer was in charge of work in the field.

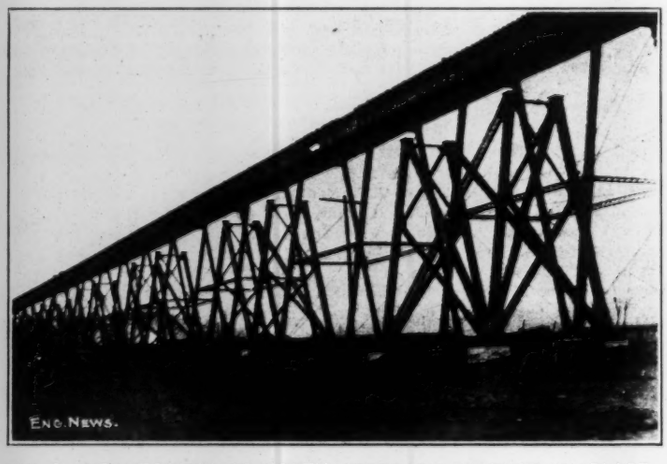

FIG. 8. NEW TOWERS OF NORTH APPROACH VIADUCT IN PLACE, POINT PLEASANT BRIDGE.

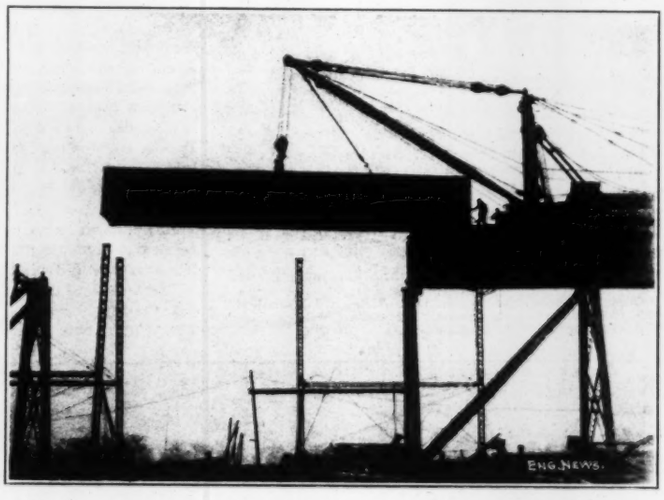

FIG. 9. SETTING A 60-FT. GIRDER OF THE NORTH APPROACH VIADUCT, POINT PLEASANT BRIDGE.

THE FAMOUS BEAR VALLEY DAM, California, is to be replaced by a new and higher dam. The various parties in interest have engaged Messrs. G. O. Newman, James D. Schuyler, M. Am. Soc. C. E., and F. C. Finkle to investigate and report on the subject. The original Bear Valley Dam was described and illustrated in Engineering News for April 7 and June 23, 1888, and Nov. 23 and 30, 1889.

Civil Service Examination Questions.

The following questions are reproduced from civil service examinations given recently by the New York State Civil Service Commission. Although not verbatim copies they are correct in the main.

New York State Civil Service Examination, July 31, 1909.

ASSISTANT CIVIL ENGINEER, SALARY $1,201 to $1,800.

Time allowed, 8 hrs.

v (1) State the theory of the Venturi meter and explain how the flow can be measured having the area and gage readings.

v (2) What are the adjustments of the transit and level? Give the order in which they are made. Explain how they are made. Which, in your opinion, is the better method of measuring an angle, by continued readings or by multiplying the readings?

Explain what is meant by double hubbing.

v (3) What is the maximum moment that the following will carry—a plate 15 × ⅝-in. with a single 4 × 4 × ½-in. angle riveted to the upper or compression side. Length, 20 ft.

v (4) If the above is loaded uniformly 200 lbs. per lin. ft., compute the maximum shear in the rivets if there are 5 on each end.

v (5) Design in detail and give full computation for a reinforced-concrete footing for two columns 10 ft. apart, one carrying 20 tons and the other 70 tons. Depth given = 3 ft.

x (6—7) Design in detail and give full computations for the timbering for a trench cut in a paved street. Trench, 30 ft. deep; first 15 ft. in ordinary earth, and below this wet clay and gravel. Trench, 30 ft. wide.

x (8) Write full instructions for inspectors on track laying, placing ballast, tunneling in rock, earth and quicksand.

x (9) In a triangulation survey two points are inaccessible and the instrument is set up at two points. Angle A—B—C = 38° 10′ 0″, Angle B—A′—C = 110° 20′ 40″, Angle B—A′—A = 136° 0′ 10″, distance A—A′ = 26.728 ft., distance A′—B = 5297.630 ft. Find distance B—C.

v (10) A combined sewer is 6 ft. in diameter, slope 1 in 2,000; compute the flow when running half full and give your estimate of the area this sewer would drain if it were in Manhattan.

v = No text books used.
x = Text books may be used.

New York State Civil Service Examination, July 31, 1909.

ASSISTANT CIVIL ENGINEER, SALARY $1,801—$2,400.

Time allowed, 8 hrs.

First paper—Answer all questions. No text book.

(1—2) Design steel tower 50 ft. high for single track railroad bridge, 60 ft. between towers, 30 ft. between bents, track carried directly on deck girders 3 ft. and 6 ft. in depth, 7 ft. c. to c. Assume uniform rolling load 5,000 lbs. per lin. ft. Write specification for the structure and upon it base the design. Show in full detail connection between girders and post of tower.

(3—4) Reinforced-concrete retaining wall 20 ft. high, 2 ft. 6 ins. thick at top, 4 ft. thick at bottom, front face vertical—surcharge on 1 on 1½ slope to height of 6 ft. above top of wall. Sand and gravel top 15 ft., below that wet clay. Allow for a vertical load of 10,000 lbs. 16 ft. from face of wall. Complete design of section, showing method of building foundation; figure reinforcement required; use no counterforts.

Second paper—Answer any 3 questions. Text books may be used.

(5) A sewer 1,500 ft. long is to built in a street, to carry from 60 to 600 cu. ft. per sec. with a drop of 4 ft. invert at upper end 17 ft. below street level; street level upper 10 ft. filled ground, below that heavy clay with pockets of quicksand—sewer is to be carried under these two streets containing pipes, tracks, etc., under these two streets and between them, for a length of 600 ft., sewer is to be built in tunnel. Design sewer sections both in open cut and in tunnel and show sketch of outlet of sewer discharging freely into river at end of wooden dock.

(6) Describe construction of sewer—estimate force and plant required and figure cost of job.

(7) Describe surveys necessary for sewer, preliminary and during construction; organization of party for each case; nature of work; results to be obtained; points to be set, where and how; precautions to be taken.

(8) Write specifications for sewer.

(9) Describe method of valuation of property and franchise of a water-works company pumping 6,000,000 gals. daily from a river to a distributing reservoir and supplying 5,000 people.

(10) Suppose available a force of 20 men, both office and field men, and it is necessary to make a report, with plans and specifications, in 90 days, on eliminating a grade crossing, describe schedule of operations, organization of parties, work performed by each throughout the available time; data to be obtained and all steps in the work.

(2) What is Portland cement? What is natural rock cement? Explain the complete process of manufacture of each.

(3) Name in order all the tests of cement, giving your standards of requirements for each.

(4) What are the points to be covered in writing specifications for a macadam road? Give a complete outline, mentioning the points in order.

(5) What is a pneumatic caisson? What are the uses of the different parts, and under what circumstances would a pneumatic caisson be used?

(6) What are the elements determining the depth of excavation for a foundation? Name in order of their bearing value, the different soils which may be used. A test boring shows the following. Loam, 2 ft.; fine sand, 12 ft.; water-bearing gravel, 4 ft.; rotten rock, 3 ft.; solid rock. How deep would you go for the foundation of a highway bridge over a railroad? Why?

(7) What are the different methods of waterproofing concrete in common use? What is the effect of their use upon the concrete?

(8) What is the prismoidal formula? How would you prepare a table for its use in calculating earthwork? Illustrate.

(9) Compare wooden and concrete piles as to economy and utility. What are the different kinds of concrete piles and how are they placed? How could concrete piles be used to avoid difficult and expensive caisson or coffer-dam work?

(10) What is the elements determining the depth of excavation for a foundation? Name in order of their bearing value, the different soils which may be used. A test boring shows the following. Loam, 2 ft.; fine sand, 12 ft.; water-bearing gravel, 4 ft.; rotten rock, 3 ft.; solid rock. How deep would you go for the foundation of a highway bridge over a railroad? Why?

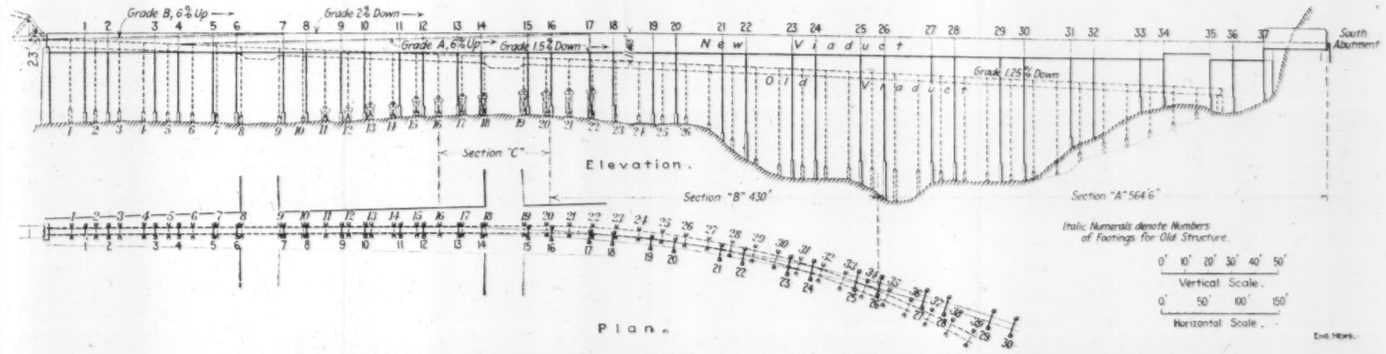

FIG. 11. DIAGRAM PLAN AND ELEVATION OF SOUTH VIADUCT, SHOWING SEQUENCE OF ERECTION.

(2) Find by the use of logarithms, the value of

$$493 \times 8.47\ (7.134 + 0.42 \times 76.1^2)^{\frac{1}{2}}.$$

(2) Two dams are connected by a 36-in. pipe. The pipe is 4,670 ft. long and the difference in elevation of the two water surfaces is 210 ft. Allowing for loss at entry but not for any loss due to curvature, what will be the discharge?

(3) A circular aqueduct is 5 ft. in diameter, has walls 1 ft. 6 ins. thick, and the radius on the center line is 2,000 ft. How many cubic yards of masonry in a portion the central angle of which is 27° 9′?

(4) A lock wall 26 ft. high is 3 ft. wide on top. What must be the bottom width to insure stability against a backing of sand and gravel level with the top, when the lock is empty?

(5) A beam of 20-ft. span is to carry a uniform load of 1,000 lbs. per lin. ft. and a load of 3.5 tons at the middle. The beam is to be made of two timbers each 8 ins. wide, one on either side of a steel plate. Using 29,000,000 lbs. as the modulus of elasticity of the steel; 850,000 lbs. as that of the timber; 16,000 lbs. as the unit stress in the steel, and 1,250 lbs. as that in the timber, design the beam, using commercial sizes of timber.

(6) The run-off from a watershed is 300 cu. ft. per sec., and is to be carried partly through a rock tunnel and partly in an open ditch whose side slopes are 1 on 1½. The slope in the tunnel is to be 5 in 1,000; in the ditch, 1 in 1,000. The bottom width in either case is to be 10 ft. What will be the depth and the velocity in each section?

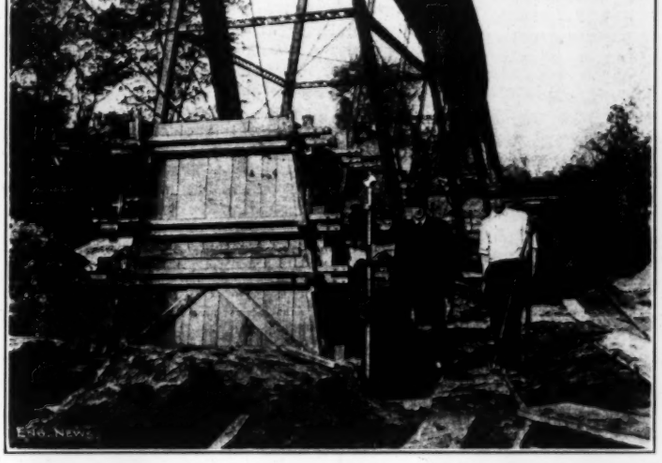

FIG. 10. BUILDING TOWER FOOTINGS FOR NEW SOUTH APPROACH VIADUCT OF POINT PLEASANT BRIDGE.

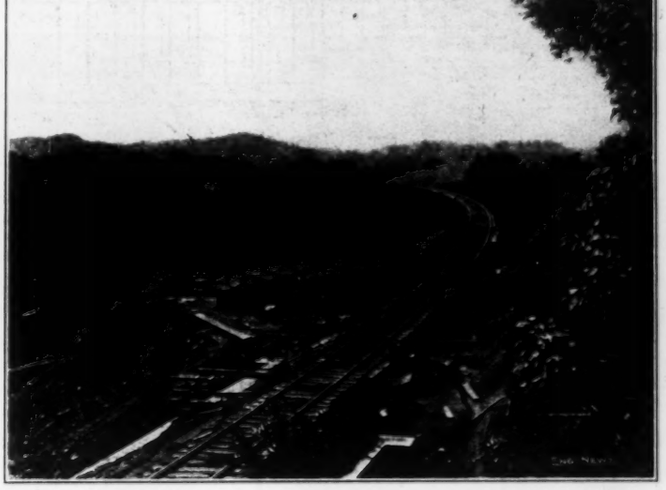

FIG. 12. THE NEW SOUTH APPROACH AND VIADUCT, POINT PLEASANT BRIDGE.

2½° curve and 0.2% grade (0.3% compensated) substituted for 3° curve on 1¼% grade (1½% compensated.)

New York State Civil Service Examination, April 18, 1908.

ASSISTANT CIVIL ENGINEER, SALARY, $5 TO $6 PER DAY.

First paper; answer all questions.

(1) How would you make a survey of a watershed area? State everything that would be necessary to furnish complete information for a report.

Second paper. Answer all questions. Reference books may be used.

A NEW TIE-TREATING PLANT for the Pennsylvania R. R. is to be built at Point House Pier, Greenwich Point, Philadelphia, Pa. This new plant will be of one cylinder, similar to the large one now in operation by the same railway at Mt. Union, Pa., except that it will be designed to accommodate an additional cylinder when needed. The capacity of the new plant will be from 750,000 to 1,000,000 ties per year, on day and night shifts and with the two plants running the railway will be able to treat about 1,500,000 ties each year. In connection with the Greenwich Point plant, there is to be a storage yard of 25 acres, affording room for a year's product of ties. Provisions have been made for the treatment also of bridge timbers, poles and other wood used by the company.

BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION are being studied by a special committee of the American Society of Civil Engineers. A circular stating the aim of the committee may be secured from Mr. A. H. Blanchard, Secretary, East Side Station, Providence, R. I. There may also be obtained, we assume, copies of a detailed schedule designed to bring before the committee "data concerning the use of bituminous materials in road surfacing"; also, a "list of analyses and methods of testing bituminous materials, proposed by committee."

Guard Rails and Deck Construction for Railway Bridges.

In the design of the deck and track construction of railway bridges, attention must be given to the necessity for providing against accidents in case of a derailed train coming upon the bridge, the main idea being to carry such a train safely across the structure to the comparative safety of a firm roadbed on the approach. The usual practice is to provide outside guard timbers (whose chief value is to maintain the positions of the ties) and inside guard rails, but the details of this practice vary considerably. The typical arrangement is shown in Fig. 1, which represents the practice of the Lehigh Valley Ry. We have made inquiry as to the details of practice of a number of railways in this respect, and the results of this inquiry are given in the present article and the accompanying tables.

TIES.—Bridge ties are usually of yellow pine or white oak (the former being the less liable to warp), but Oregon pine or Douglas fir is used by some roads. The ties are sawed, and a very general size is 8 × 8 ins. With widely spaced stringers the depth may be increased, the maximum section being 8 × 16 ins. The length ranges from 9 ft. to 13 ft.; greater lengths (14 ft. or 15½ ft.) are used sometimes for every fourth tie in order to carry a plank walk. For double track, some roads used timbers 24 ft. long to carry both tracks, but the practice is not general. The ties are frequently spaced 6 ins. apart in the clear; some roads use 4 ins. (which is preferable) and others use 8 ins.

The ties are usually held in position (to prevent displacement or bunching by derailed wheels) by boxing out the guard timbers over the ties. A very few roads use spacing blocks between the ties for the same purpose. This practice has been adopted in some cases by the Pittsburg & Lake Erie Ry., three lines of 6-in. blocks being placed between the ties (at the middle and at each end). On ballasted floors, as a rule, no special means are provided to hold the ties, the ballast being relied upon to prevent their displacement.

On steel bridges, the ties are usually boxed ½-in. or 1 in. on the bottom to fit the flanges of the girders or stringers. This holds them in line laterally, but makes tie renewals more difficult, as the track must be raised sufficiently to clear the flanges. When the ties are not boxed, they can be slipped out of place as soon as the fastenings are removed. In some cases (as on the Chicago & Northwestern Ry.) the ties are not boxed, and the stringer flanges are so formed as to have no top rivets or cover plates, thus giving a flat seat for the ties. In other cases (as on the Louisville & Nashville Ry.) the stringer or girder webs extend slightly above the flange angles, and the ties are grooved to fit over the ribs thus formed.

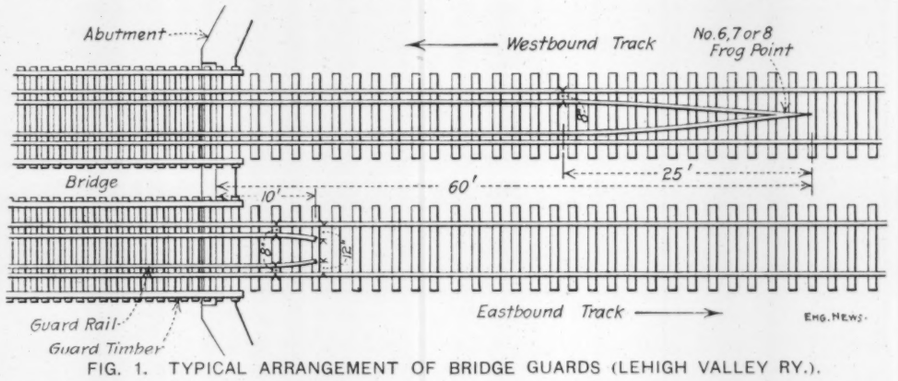

FIG. 1. TYPICAL ARRANGEMENT OF BRIDGE GUARDS (LEHIGH VALLEY RY.).

As a rule every third or fourth tie is fastened to the structure. Large spikes, lag screws or through bolts are used on wooden bridges and trestles. On steel bridges, hook bolts are commonly employed, but some roads put bolts through the flanges of the stringers or girders. The principal purpose of the fastenings is to keep the deck (and track) in line. But where high winds or cyclones may occur, or where floods may rise above the track level, the fastenings must provide for anchoring the deck securely to the structure.

OUTSIDE GUARD TIMBERS.—Guard timbers outside the track rails are commonly employed. When designed to hold the wheels of derailed cars in line and guide them across the bridge, they are placed usually from 10 to 15 ins. from the gage side of the track rails. On some roads, however, they are regarded only as tie spacers, and are put at the ends of the ties. The Chicago & Northwestern Ry. bridge over the Mississippi River at Clinton, Ia., has the guard timbers on the ends of 12-ft. ties, but the spacing between the trusses is made sufficient to clear a car whose wheels have reached the guard timbers.

The almost universal size is 6 × 8 ins. (laid flat), with the bottom boxed out for the ties. In some cases, the boxing is sufficient to bring the top level with the rail heads, but this is not good practice unless the timbers are used merely as tie spacers. The timbers usually have horizontal scarf joints, with a vertical bolt through the scarf and tie; the Southern Pacific Ry., however, makes the scarf vertical and puts two horizontal bolts through the joint. Butt joints are used on some roads. The timbers are fastened to the ties by through bolts. The nuts and washers are usually on top, but some roads have the bolt heads on top and fitted into cup washers so that there is no projection above the top of the guard timber. Lag screws are sometimes used as fastenings, either alone or alternating with bolts.

Until a few years ago it was general practice to extend the guard timbers on the approach embankments, flaring them outward and supporting them on long ties, so as to catch and guide derailed wheels. This practice has been abandoned almost entirely, the guard timbers now ending with the bridge floor, and the inside guard rails being extended to serve the same purpose.

INSIDE GUARD RAILS.—It is general practice to put a guard rail inside each track rail, in order to check and guide derailed wheels. These are much more effective than the outside timbers, for reasons which are discussed elsewhere. The inside guards are usually steel rails, though a few railways use timbers. Old rails may be used, but should not be much lower than the track rails. If it is desired to use old light rails they may be raised to the proper height by cast-iron chairs or pedestals. On ordinary railways, the guard can be only a little (if at all) above the

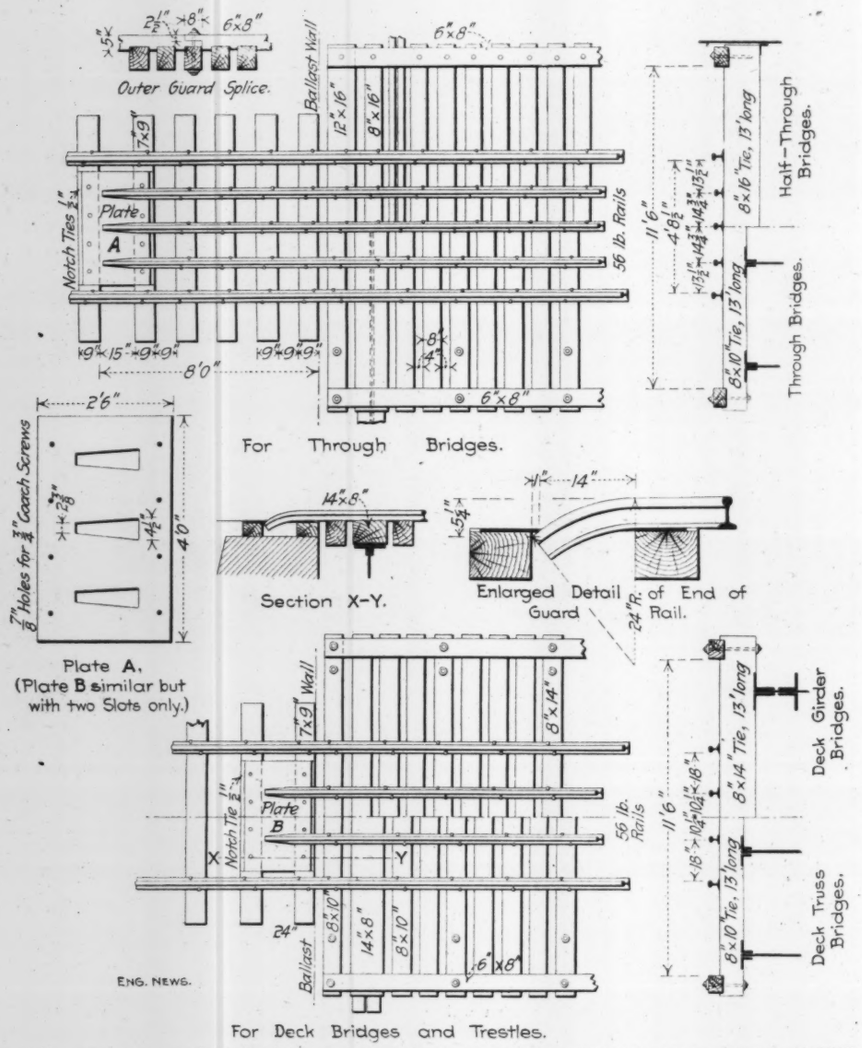

FIG. 2. DECK AND GUARD RAIL CONSTRUCTION (WITH DOUBLE AND TRIPLE INSIDE GUARDS); CANADIAN PACIFIC RY.

TABLE I.—GUARD RAILS ON RAILWAY BRIDGES HAVING OPEN FLOOR CONSTRUCTION.

Railways	Outside Guard Timbers: Size ins.	Boxed ins.	Kind of wood	Distance from gage side of rail ins.	Fastened to ties	Inside Guard Rails: Kind of guard	Distance from gage side of rail ins.	Fastened to ties	Extension on Approach: Total length ft.	Curved length ft.
A. T. & S. F.	6 x 8	.....	.....	17¼	Hook bolt ev. 3d tie	Old rails	8	Spiked each tie	30 or 60	15 or 30
Balto. & Ohio	6 x 8	1	L.-l. yell. pine	13 min.	¾-in. bolts, boat spikes	Old rails	10	Spiked alt. ties on bridge, every tie on app.	61	48
Bos. & Me.	6 x 8	1	L.-l. yell. pine	On ends of ties	⅝-in. lag s. every 4th tie	Old rails	9	Spiked	57	27
Can. Pac.	6 x 8	.....	Pine	40¾	Bolt every 4th tie	56-lb. rails, 3 for thro. / 56-lb. rails, 2 for deck	13½ / 18	Spiked	8 for thro. / 2 for deck	None
Car., C. & Ohio	6 x 8	1	L.-l. yell. pine	20¾	Bolt every 4th tie	60-lb. rails	8	Spiked	100	30
Chi. & E. Ill.	6 x 8	.....	Yell. pine	17¾	Bolt every 3d tie	Same as track	9	Spiked	49½	30
Chi., B. & Q.	6 x 8	Box	.....	22½	Lag screw ¾ x 10 ins.	Nearly same as track	9½	Spiked	90	....
Del. & Hud.	6 x 8	1	Oak	8 + railhead	¾-in. bolt every 3d tie	Same as track	8	Spiked	30	....
Leh. Vall.	6 x 8	1½	L.-l. yell. pine	21¾	¾-in. bolt every 3d tie	60 to 80-lb. rails	8	Spiked every tie	60	25
Lou. & Nash.	9 x 5 / 12 x 5	1	Yell. pine	10 + railhead	Lag screw alt. ties	Timbers 4 x 9 ins., boxed ¾-in.	3½	Lag screw alt. ties	.....	....
Mich. Cen.	6 x 8	Not	W. oak	34	¾-in. bolt every 3d tie	3 rails, 60 to 80-lb.	14½	Spiked every tie	75	....
N. Y., O. & West.	6 x 8 (to 20 ft. beyond bge.)	.....	Yell. pine	21¾	¾-in. bolt alt. ties	Rails	8	Spiked every tie	44	36
Penn. Lines	6 x 8	1½	L.-l. yell. pine	13 to 15	Bolt every 4th tie	Rails	10	Spiked ev. tie, rail braces	200	14
Penn. Ry.	6 x 8	.....	.....	10	.....	Same as track	10	Spike alt. ties (ev. tie on curve)	61	48
So. Pac.	6 x 8	1½	Ore. pine (not creos.)	16½	Hook bolt every 3d tie	Old rails, 50 lb. and over	8 (3 with rerails)	Spiked every tie, rail braces	50 (75 with rerails)	20
Wabash	6 x 8	1	Yell. pine	18 min.	Lag screw every 3d tie	Old rails	8	Spiked every tie	60	30
Chic. So. S. Elev.	6 x 8 (on edge)	.....	... ...	10	¾-in. bolt alt. ties	Timber, 6 x 6 ins. Rail for inside of curve.	4	Bolt alt. ties	.....	....

track rail; on a high-speed electric railway in Germany, however, it rises some inches above the latter, the guard rail being laid with its web horizontal (resting on chairs) and its base facing the track rail.

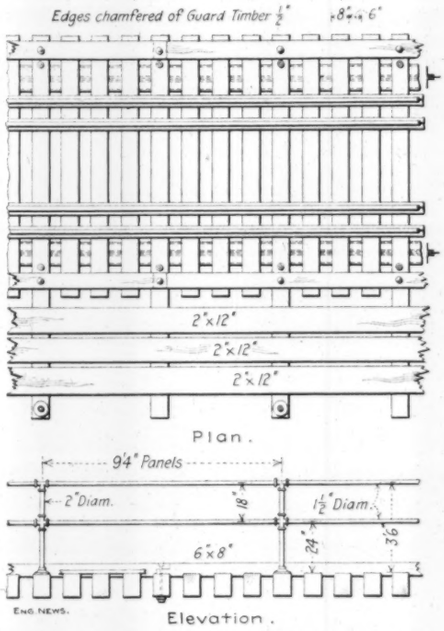

Fig. 3. Bridge Deck with Sidewalk; Carolina, Clinchfield & Ohio Ry.

The clear distance from the gage side of the track rail to the face of the guard rail is usually from 8 to 10 ins., with a minimum of 3½ ins., and a maximum of 13 or 14 ins., but the wide spacing is not to be recommended. Where rerailing devices are used on the Southern Pacific Ry., the spacing is only 3 ins. The rails are spiked to the ties, and sometimes reinforced by rail braces. Some railways use the Jordan guard, which consists of three lines of rails between the track rails (one in the middle and the others 13½ ins. from the track rails.)

Steel angles are sometimes used instead of ordinary rails on large bridges. On some bridges of the Chicago & Northwestern Ry. the horizontal leg of the angle is towards the track rail. A heavy plank is laid against the back of the vertical leg, and a similar plank just outside of the track rail. In case of derailment, the flanges of the inside wheels would ride on the guard angle while the treads of the outside wheels would ride on the plank. In other cases, as on the Thebes bridge, the horizontal leg of the guard angle is towards the middle of the track, allowing derailed wheels to ride on the ties.

The inside guard rails are extended some distance beyond the bridge, and then brought together at a point, in order to catch derailed wheels and guide them into position for crossing the bridge in safety. The ends of the rails may be fitted to a casting or old frog point, or they may be bent downward. The rails spreading outward from the point should be reinforced by rail braces. The usual distance from the bridge to the point is about 60 ft., sometimes as little as 30 ft. (which is too short) or as much as 200 ft. With a 60-ft. length, the rails are straight for 30 ft. and then incline towards the point. With the Jordan guard (noted above), the rails extend parallel with the track for 8 to 15 ft. beyond the bridge, and have their ends bent down and passed through a slotted plate spiked to the ties. On double-track bridges, the guard rails at the leaving end of each track need extend only a few feet beyond the bridge.

Details of Practice on American Railways.

In the accompanying tables are given the details of practice of several leading railways in regard to their bridge deck and guard rail construction, and further particulars as to these matters are contained in the following notes compiled from information furnished by the railways.

ATCHISON, TOPEKA & SANTA FE RY.—The ties are sized to 7¾ ins. in depth, and the guard timbers are boxed to bring their tops level with the rail heads. On wooden bridges the deck is

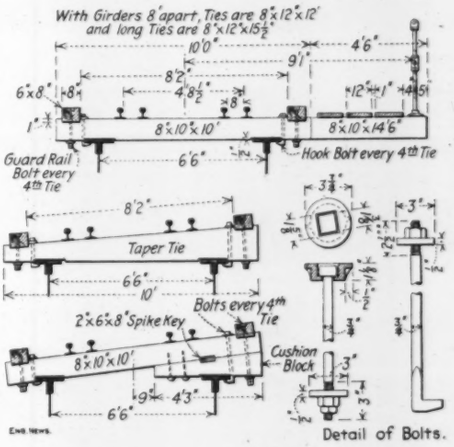

Fig. 4. Bridge Deck Construction on Curves; Carolina, Clinchfield & Ohio Ry.

secured to the structure by passing the guard-timber bolts through the ties and stringers. On steel bridges, these are hook bolts which pass through the guard timbers and ties and grip the flanges of the girders. The inside guard rails are old rails (not more than 1 in. shallower than the track rails), set with the smooth side facing the track rails. They are full spiked to each tie and have all joints full bolted. No guards have been

TABLE II.—DECK CONSTRUCTION AND GUARD RAILS ON RAILWAY BRIDGES.

Railways	Ties: Section ins.	Length ft.	Kind of wood	Clear spacing ins.	Deck fastenings to: Wooden structures	Steel Structures	Deck complete per lin. ft. including track: Weight lbs.	Cost $	Guard rails used on: Ballasted floor bridges	Masonry bridges
A., T. & S. F.	8 x 8	12	L.-l. y. pine and Ore. fir (treated and untreated.)	6 max.	Through bolts	Hook bolt every 3d tie	360 to 400	2.60 to 3	None	None
B. & O.	8 x 8 to 8 x 16	9 min.	L.-l. y. p., W. oak	6	Boxed 1 in. over stringers	Boxed 1½ in. to 1½ ins. over flanges	400 min.	4.50	Inside, on curves	Inside, on curves
Bos. & Me.	6x8 for wood bges. / 7x8 for steel bges.	12	L.-l. y. pine; spruce	6 (2 in yards)	Spike every 4th tie to stringers	Ties boxed ½ in. for flanges	332	3.00	None	Only where bridge is narrow
Can. Pac.	8 x 8 to 8 x16	13	Pine	4	Hook bolt to stringer	every 4th tie	580 to 800	7 ave.	Ins. and outside	Ins. and outside
Car., C. & O.	8 x 10 / 8 x 10	10(6½ ft.spac.) / 12(8 ft.spac.)	L.-l. yell. pine, some oak	6	¾-in. bolts	Ties boxed; hook bolt every 4th tie	450 6½ ft. spac.) / 550(8 ft. spac.)	3.50 / 4.00	Same as on open deck for spans over 25 feet	
Ch. & E. Ill.	8 x 8 / 8 x 10 / 8 x 12	9 / 11 / 12	Yellow pine or oak	4	Spike every 4th tie to stringers	Boxed over flanges	556	6.93	None	None
Ch., B. & Q.	8 x 8 for 6 ft. spac. / 8 x 10 for 7 ft. spac. / 8 x 12 for 8 ft. spac.	10 / 10 / 10	W. pine or fir	4	¾-in. lag screw to stringers	¾-in. bolt through flanges	.....	.....	Inside	Inside in some cases
Del. & Hud.	9 x 9 and 9 x 11	9	Yellow pine	6	.....	Boxed over flanges	.....	.....	Neither	Neither
Leh. Vall.	8 x 9	10	L.-l. yellow pine	6	Spike every 3d tie	Hook bolt every 6th tie	.....	.....	Neither	Neither
L. & N.	8 x 8 for 6½ ft. spac. / 8 x 9 for 7 ft. spac. / 8 x 11 for 9 ft. spac.	10 / 10 / 13	Yellow pine	6	.....	Grooved ¾ in. for web plate; hook bolt every 3d tie	.....	.....	Neither	Neither
Mich. Cen.	6 x 10 on wood / 8 x 10 on steel / 6 x 8 on wood / 9 x 10 on steel	12 / 12 / 9 / 10	W. oak, Ore. fir. / Yellow pine	4 / 8	¾-in. bolt every 3d tie / Boxed 1½ in.	Hook bolt every 3d tie / Boxed 1½ in.	243 / (350 (ex. of rails)	1.61 / 3 50(ex. of rails)	Inside / No ball. floors	Inside for spans over 50 ft. / No mas. bridges
N. Y., O. & W.										
Pa. Lines	8 x 8 min. depth	9 min.	L.-l. pine, oak	8	Bolted	Boxed; clip bolt every 4th tie	400	2 ave.	Inside, if necessary	Inside, if necessary
Pa. Ry.	Varies with girder spacing		Hard yellow pine	6	Bolt every 3d tie	Bolt every 3d tie	.....	.....	Inside	Inside
So. Pac.	8 x 10	10	Ore. pine (creos.)	4	Bolts and boat spikes	Boxed; hook bolt every 4th tie	405 actual / 500 for estim.	.....	Neither	Neither
Wabash	8 x 8	10	.....	8	Lag screw every 3d tie	Boxed and hook bolts	400	7	Neither	Neither
Ch. S.S. Elev.	6 x 8	8 on tan	.....	18 c. to c.	.....	Hook bolts alt. ties	.....	.....	.....	.....

used on ballasted floor bridges, but it is thought that inside guard rails should be used on all bridges. This road has two unballasted solid-floor steel bridges (on the Chicago track elevation work), and these have inside guard rails of the same weight as the track rails. The truss bridges on the Valley Division in California (formerly the San Joaquin Valley Ry.) have

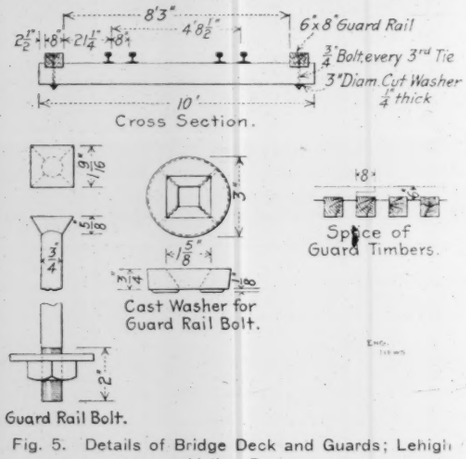
6"x8" Guard Rail
3/4" Bolt every 3rd Tie
3" Diam. Cut Washer 1/4" thick
Cross Section.
Splice of Guard Timbers.
Cast Washer for Guard Rail Bolt.
Guard Rail Bolt.

Fig. 5. Details of Bridge Deck and Guards; Lehigh Valley Ry.

Latimer rerailing guards; they have also bumping posts of timbers (14 or 16 ins. square) on the approaches, set in line with the trusses and opposite the point of the inside guard rail. The use of both rerailing guards and bridge bumping posts is now almost obsolete, as noted elsewhere.

BALTIMORE & OHIO RY.—The ties are in size as follows: 8 × 8 ins. where the stringers or girders are spaced 6½ ft. c. to c.; 8 × 10 ins. for 7 ft. and 8 ft. spacing; 8 × 12 ins. for 9 ft.; 8 × 14 ins. for 10 ft., and 8 × 16 ins. for 12 ft.

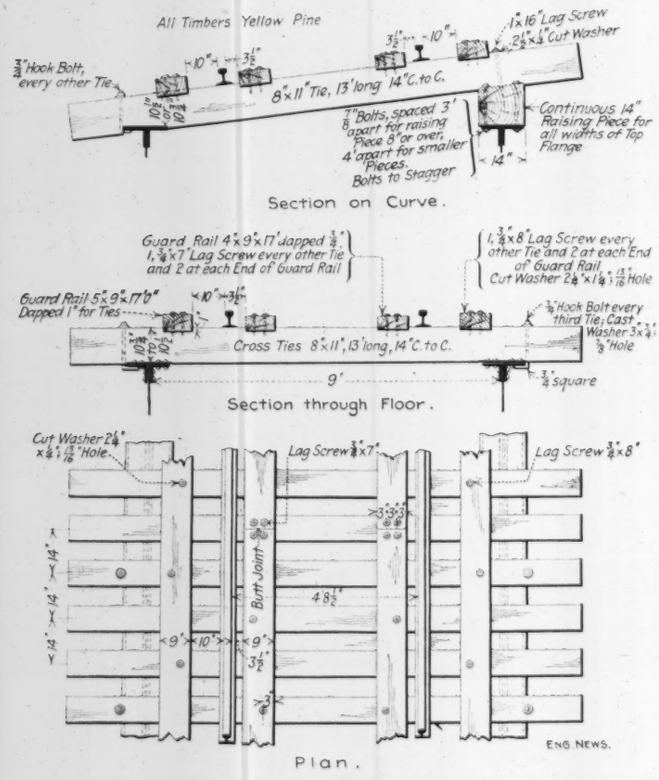
All Timbers Yellow Pine
Hook Bolt, every other Tie
8"x11" Tie, 13' long 14" C. to C.
Bolts, spaced 3' apart for raising Piece 8" or over, 4' apart for smaller Pieces. Bolts to Stagger
Continuous 14" Raising Piece for all widths of Top Flange
Section on Curve.
Guard Rail 4"x9"x17' dapped 3/4" Lag Screw every other Tie and 2 at each End of Guard Rail
Guard Rail 5"x9"x17'6" Dapped 1" for Ties
Cross Ties 8"x11", 13' long, 14" C. to C.
Hook Bolt every third Tie; Cast Washer 3"x3"x3/8" Hole
Section through Floor.
Cut Washer 2¼" x ⅜" x 13/16" Hole
Lag Screw ¾"x7"
Lag Screw ¾"x8"
Butt Joint
ENG. NEWS.
Plan.

FIG. 6. BRIDGE DECK WITH TIMBER GUARDS; LOUISVILLE & NASHVILLE RY.

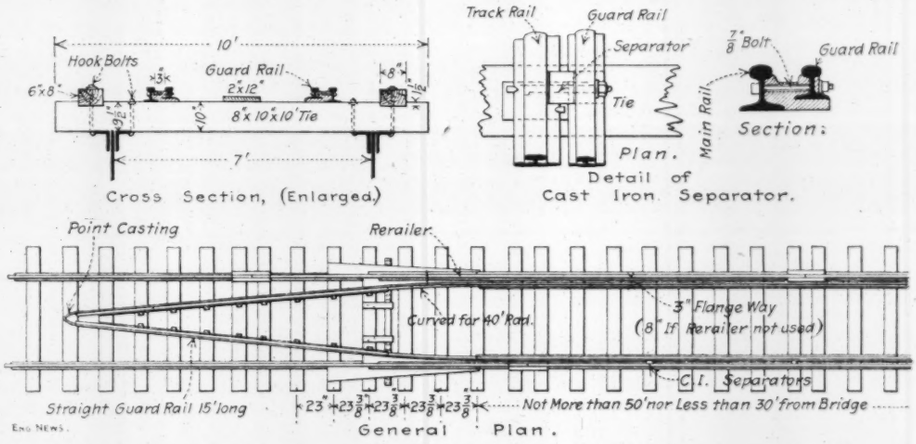

FIG. 8. BRIDGE GUARDS WITH RERAILING DEVICES; SOUTHERN PACIFIC RY.

joints are full bolted. At the ends they are brought together till the heads are 4 ins. apart, and are bent down so that their tops are below the tops of the ties. Inside guard rails are required on all through bridges and in tunnels, on all deck structures having open floors, and on arches and ballasted floor bridges when these are on curves. The outside guard timbers have 6-in. scarf joints, over the ties, with a ¾-in. bolt through the splice and tie. At the tie on each side of this joint an extra ½-in. boat spike 10 ins. long is driven through the guard timber. Every intermediate tie has a ¾-in. bolt or ½-in. boat spike at each end; the bolts and spikes alternate, so that they are both four ties apart. They are driven in staggered lines. The bolts have 2½-in. mushroom heads, with two fins on the under side which bite into the wood and prevent the bolt from turning with the nut. The square nut and 3-in. washer are at the bottom of the tie. The guard timbers are so placed that the washers are adjacent to the outer edges of the flanges of the girders.

On curves, beveled ties are used when the stringers, and ½-in. to 1½ ins. over girder or stringer flanges. Two depths of ties are used on girder spans when the thickness of cover plates is such that the boxing would be more than 1½ ins.

The inside guard rails are smooth relaying rails. They are spiked to alternate ties on the bridge and to every tie on the approaches. The thickness of the larger end does not exceed 16 ins., the thickness under the inner rail being not less than that for ordinary ties. Where the inclination of the tie would require a greater thickness than 16 ins. at the butt, ordinary ties are used, elevated by wedge-shaped blocking pieces on the outer girder, and flattened to give an even bearing on the inner girder. The arrangement is very similar to that shown in Fig. 4.

When the curve does not exceed certain limits (varying with the length of bridge)* the ties are laid with ends parallel with the girders. Beyond these limits, they are laid with ends parallel with the track. In this case they are ordered of such length as to project at least 6 ins. beyond the edge of the girders; that is, the length of tie is equal to the distance over supports (out to out), plus the ordinate of curve and plus 12 ins.

The weight of deck varies with the size of ties used, which depends entirely on the distance center to center of supports and the superelevation of the outer rail. In calculations, the minimum weight assumed is 400 lbs. per foot of single track to cover the weight of the ties, guard-rails, track rails and fastenings. The cost per foot varies with the size of the tie used, but the decks cost about $45 per 1,000 ft. B. M. in place.

BOSTON & MAINE RY.—The ties are mainly of long-leaf yellow pine, but spruce ties are used in a few cases for covered wooden bridges in northern New England. They are planed on one side to give a uniform thickness, and are secured by spiking every fourth tie to the stringers. This is done to keep the track in

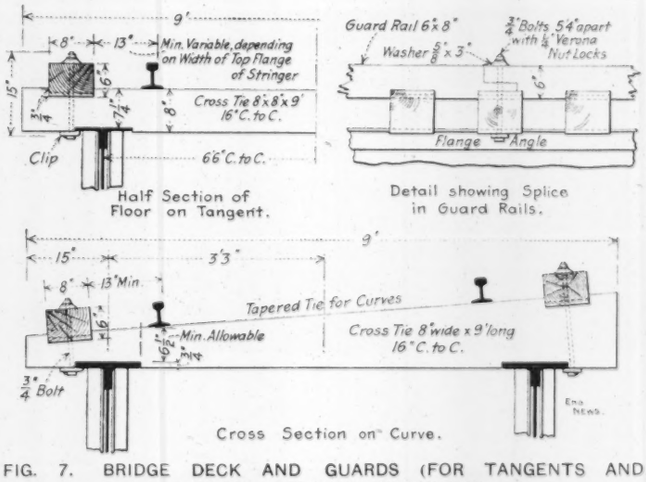
Min. Variable, depending on Width of Top Flange of Stringer
Cross Tie 8"x8"x9' 16" C. to C.
Clip
66" C. to C.
Half Section of Floor on Tangent.
Guard Rail 6"x8"
3/4" Bolts 54" apart with 1/2" Verona Nut Locks
Washer 5/8" x 3"
Flange
Angle
Detail showing Splice in Guard Rails.
Tapered Tie for Curves
Min. Allowable
Cross Tie 8" wide x 9' long 16" C. to C.
3/4" Bolt
Cross Section on Curve.

FIG. 7. BRIDGE DECK AND GUARDS (FOR TANGENTS AND CURVES); PENNSYLVANIA LINES.

*The limits are as follows: 10° for bridges up to 25 ft. long; 6° for spans of 25 to 50 ft.; 4° for 50 to 75 ft.; 3° for 75 to 100 ft.; 2° for 100 to 125 ft.; 1° for spans of 125 to 150 ft.

line, and not to hold the floor from lifting. The latter is considered necessary only in a cyclone region or where floods reach above rail level. No hook bolts or other holding-down bolts are used, as no tendency of a floor to lift has ever been noticed, and the stringers are so arranged that a derailed car does not tend to lift the ties. The ties are notched ½-in. or more over the girder or stringer or stringer flanges of metal bridges in order to hold the track in line. The clear spacing of the ties is 6 ins., but in the terminal yards at Boston this is reduced to

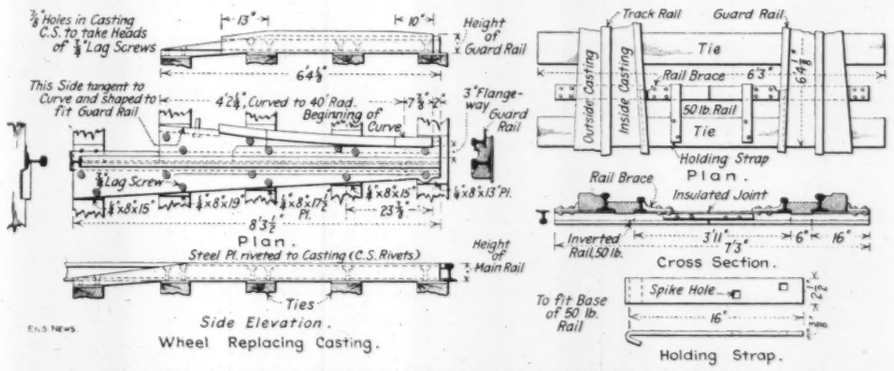

FIG. 9. DETAILS OF RERAILING DEVICES; SOUTHERN PACIFIC RY.

2 ins. The guard timbers are placed on the ends of the 12-ft. ties, and are regarded only as tie spacers. Inside guard rails are used on all bridges of over 15-ft. span, and must not be more than ½-in. below top of track rail. They are brought to a point, with the ends beveled vertically, and a wooden footguard or an old frog point is fitted in the angle.

The weight of the deck is about 222 lbs. per lin. ft. of single track for ties and guard timbers and 110 lbs. for rails and spikes (exclusive of splices), or a total of 332 lbs. The estimated cost is $3 per ft. for timber in place, but the cost is less under favorable conditions. Rerailing frogs are used at some "jacknife" drawbridges (as noted below), when the conditions are dangerous in case of derailment. For ordinary bridges it is considered better to provide floors that will safely carry derailed cars, and to let such cars run upon the floor rather than to attempt to rerail them just as they are entering the bridge.

CANADIAN PACIFIC RY.—The ties vary in section on different classes of bridges, but all

used (as shown in Fig. 2), very similar to the Jordan guard used on the Michigan Central Ry. These are lines of 56-lb. rails spiked to alternate ties. For through and half-through bridges, three lines of guard rails are used, the middle one being in the center of the track and the others 13½ ins. from the track rails. For deck truss and girder bridges and for trestles (over 100 ft. long and 25 ft. high) only two lines of guard rails are used, but these are 18 ins. from the track rails, which is an unusual spacing. The guard rails extend only 8 ft. beyond the ends of through bridges and 2 ft. beyond deck bridges; they remain parallel with the track rails but are bent down with a radius of 24 ins. so that the end is just below the level of the ties. The bent ends pass through a ½-in. slotted plate 2½ × 4 ft., which is secured to two ties by ¾-in coach screws.

CAROLINA, CLINCHFIELD & OHIO RY.—The ties are 8 × 10 ins., 10 ft. long, where the girders or stringers are 6½ ft. apart; where the spacing is 8 ft., the ties are 8 × 12 ins., 12 ft. long. Every fourth tie is 14½ ft. or 15½ ft. long (for the 6½-ft. and 8-ft. spacing respectively). These long ties carry three planks 2 × 12 ins., and each alternate one has a post for a gas-pipe hand-railing. This construction is shown in Fig. 3. The ties are mainly of long-leaf yellow pine, but white oak is used in a few cases. On steel bridges, they are boxed ½-in. on the bottom for the width of the girder flanges. The inside guard timbers are of long-leaf yellow pine; white oak is very unsatisfactory, as it warps badly. They are secured to every fourth tie by ¾-in. bolts, the heads of

ties for curves not exceeding 4° for the 6½-ft. spacing and 2° 20′ for 8-ft. spacing of stringers. Beyond these limits, beveled blocks are used, 4 ft. 3 ins. long. These are held on the outside by the tie hook-bolts and the guard-rail bolts (in every fourth tie), and on the inner side of the girder by two ½-in. boat spikes on the bottom. A similar spike is used also on the outside in the ties having no guard rail bolts. An oak key 2 × 6 × 8 ins. is driven in a groove formed by notching the tie and block. These arrangements are shown in Fig. 4. On short-span bridges with ballasted floors no guards are used, as the floor is wide. On long-span ballasted floors and on masonry bridges the same arrangement of guards is used as on open floors.

CHICAGO, BURLINGTON & QUINCY RY.—The guard timbers have butt joints instead of the more usual scarf joints. The inside guard rails are usually worn rails from curves (straightened); they are as nearly the size of the track rails as can be obtained. On unballasted solid floor bridges with steel decks (on track – elevation work) the rails are carried in troughs the inner angles of which connect with the inside guard rails. This road has had considerable experience with the Latimer rerailing guard and has found it invariably successful. Its use has been abandoned, however, for the reason that the inside guard rail (while it does not rerail the trucks) prevents wrecks quite as well as the Latimer guard so far as the bridge is concerned. This rerailing guard is considered to have two defects: (1) the throat is too narrow to allow of its being connected with an inside guard rail, so that a truck dropping off the rail on a bridge might cause a wreck; (2) there is liability of the guard being choked by ice and snow.

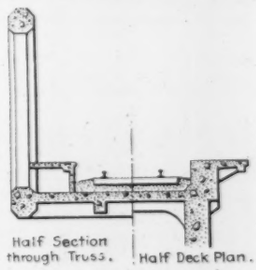

Fig. 11. Bridge Guards Forming Part of a Concrete Bridge on a French Railway.

DELAWARE & HUDSON RY. CO.—The scarf joints of the guard timbers have the vertical cuts at an angle across the timber, with a bolt through the joint and tie. The timbers are secured to every third tie with ¾-in. bolts; the heads are on top, and the bolt holes are coun-

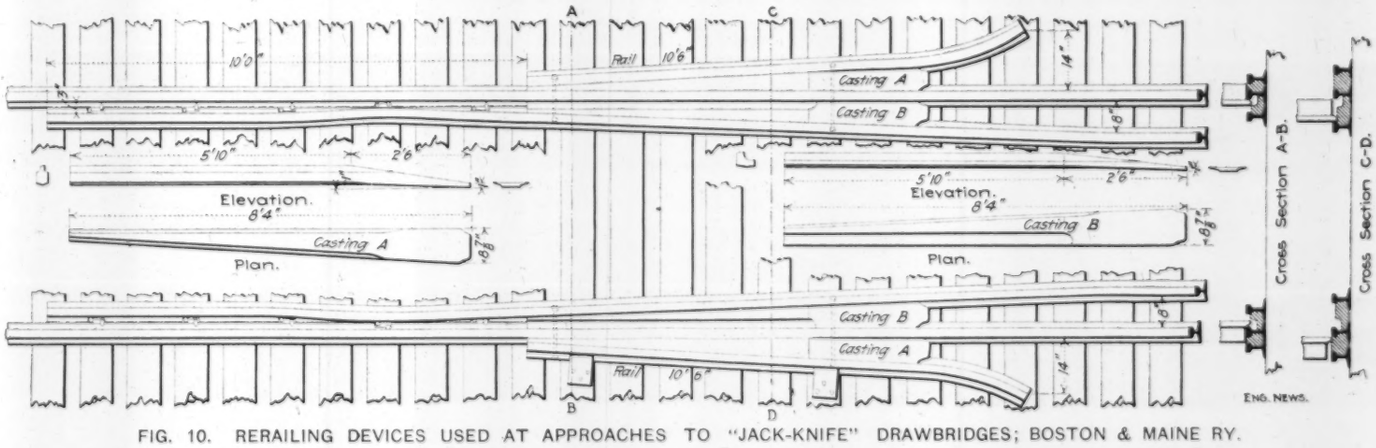

FIG. 10. RERAILING DEVICES USED AT APPROACHES TO "JACK-KNIFE" DRAWBRIDGES; BOSTON & MAINE RY.

are 13 ft. long; on trestles they are 8 × 8 ins.; on through spans, 8 × 10 ins.; on deck plate girder spans, 8 × 14 ins.; on half-deck plate girder spans, 8 × 16 ins. The weight of single track deck is 580 lbs. per ft. for through bridges, 750 lbs. for deck bridges, and 800 lbs. for half-deck bridges. The guard timbers are at the ends of the ties, and serve mainly as tie spacers. These timbers have scarf joints 8 ins. long, with a bolt through the splice and tie.

A special arrangement of inside guard rails is

which rest in cup washers let into the top of the timber (Fig. 4). The nut is screwed up against a 3-in. washer under the tie, and a follower or check nut is used to prevent slackening. The weight of the deck is 450 lbs. per ft. of track where the girders are spaced 6½ ft. apart; and 550 lbs. where they are 8 ft. apart. The cost per foot is $3.50 and $4 respectively. The sidewalk is used on all viaducts and on long-span bridges when specified.

On curves, the elevation is given by beveled

terbored to receive the bolt head and washer. This is one of the few roads extending the guard timbers upon the approach. The distance is only 8 ft., however, and the clear width between their ends is 8 ft. A standard bridge tie is placed over the bank wall and three ties 7 × 9 ins. × 10 ft. on the approach carry these timbers.

LEHIGH VALLEY RY.—The deck is held to the structure by hook bolts. The ¾-in. bolts which hold the guard timbers are of unusual design, the head being of pyramidal form and

fitting a recess in a 3-in. washer. This washer is let into the counterbored top of the bolt hole, so that the bolt head is flush with the timber. The scarf joint is 8 ins. long. The general arrangement is shown in Fig. 1, and the details in Fig. 5. At the ends of the bridge (or the entering end of a double-track bridge), the guard rails are extended 60 ft. on the approach, with a frog point fitted in the angle where the rails meet. At the leaving end of double-track bridges, they are extended only 10 ft. on the approach, being bent inward for 4 or 5 ft. so that their ends are 12 ins. from the track rails.

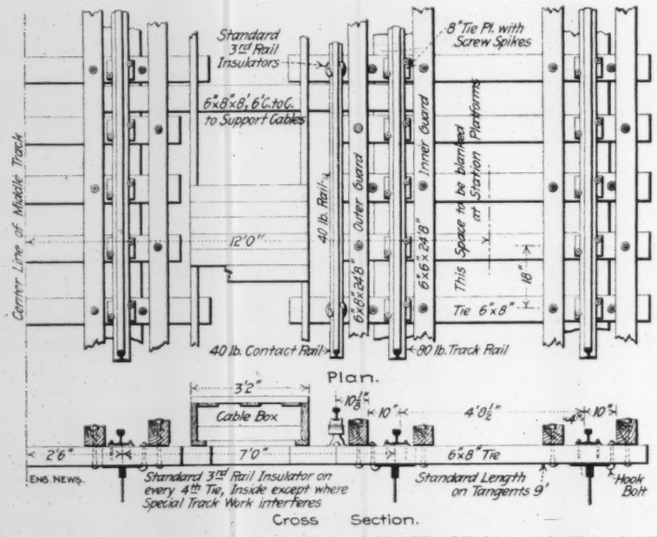

FIG. 12. DECK AND GUARD RAIL CONSTRUCTION; SOUTH SIDE ELEVATED RY., CHICAGO.

LOUISVILLE & NASHVILLE RY.—For girders 6½ ft. apart, the ties are 8 × 8 ins., 10 ft. long, with grooves ¾-in. deep to fit the webplates, which project above the flange angles. They are secured also by a ¾-in. hook bolt at every third tie, the bolt passing through the guard timber. For a 7-ft. spacing of girders the ties are 8 × 9 ins., and the outer guard timbers are 5 × 12 ins. For a 9-ft. spacing, the ties are 8 × 11 ins., 13 ft. long, and are recessed on the bottom to fit over the rivet heads of the cover-plates on the top chord. In all cases, the ties are secured by ¾-in. hook bolts in every third tie, and with the 6½-ft. and 7-ft. spacing these bolts pass through the outside guard timbers also.

Instead of steel rails for the inside guards, these guards are timbers 4 × 9 ins., laid flat, and boxed out ¾-in. for the ties. They are laid unusually close to the track rail (3½ ins). The four lines of guard timbers are in 17-ft. lengths, with butt joints; the inside guards have square joints, and the outside guards also have square joints, but the inside and outside guards break joint by 8½ ft. Each timber is secured at every other tie by a ¾-in. lag screw, in staggered lines. The inside and outside guards are secured by different ties. The lag screws are 7 ins. long for the inside guard rail and 8 ins. for the outside guards. At each joint, there are two screws in the end of each stick. The arrangement is shown in Fig. 6.

For curves, beveled ties are used, with the top horizontal beyond the outer rail, and the maximum thickness at the outer end is 14 ins. For sharp curves requiring greater elevation, a raising timber 14 ins. wide is bolted to the top flange angles of the outer girder by ⅞-in. bolts; these bolts are 4 ft. apart in blocks up to 8 ins. thick; and 3 ft. apart for blocks of 8 ins. or over. The outer end of the tie is boxed to lie horizontally on the top chord of the inner girder, as shown. Where the chord rivets have a staggered pitch of 4½ ins. (9 ins. in each row), an extra hole is put 2¼ ins. from a rivet hole to receive the ⅞-in bolt. When the pitch is less than 4½ ins., one of the rivets is omitted to make room for the bolt.

MICHIGAN CENTRAL RY.—On pile trestles, the ties are bolted to the caps with long 1-in. bolts, and every third tie has a ¾-in. bolt through the guard timber and a jack stringer directly under it. On steel bridges, the ties are boxed ½-in. for the flanges, and every third tie has a ⅞-in. hook bolt. The outside guard timbers are not boxed over the ties but are fastened to every tie. A ¾-in bolt is used at each third tie, and a 1-in. drift bolt 10 ins. long in the intermediate ties. These (in conjunction with the close spacing of the ties) are relied upon to keep the ties in place in case of derailment. The timbers are in 14-ft. lengths with butt joints. The ties have bored holes 7-16 × 4¼ ins. for all spikes.

The inside guard is of the Jordan type, with three lines of rails (60, 65 or 80-lb.); the middle one is in the center of the track, the others are 13½ ins. from the track rails. All are full bolted to every tie. The ends of the rails are bent down vertically for a length of 3½ ft., so that the top is just below the adjacent tie. The end is then cut horizontally to rest on a tie. These bent ends pass through slotted holes in a steel plate 5 × 4 ft., which rests upon four ties and secured by spikes. The arrangement is very similar to the triple-rail inside guard of the Canadian Pacific Ry. (described above), except as to the detail of the ends of the rails, the greater length of the bend necessitating cutting the tops of two ties.

NEW YORK, ONTARIO & WESTERN RY.—This road no longer uses the rerailing guards for which it was once notable, and we are informed that the inside guard rails have taken derailed trains across the bridge without any difficulty. It certainly seems, however, that it would have been better to retain the rerailers and let the track rails carry the trains in the normal way. The old style of extended guard timbers are used, extending 20 ft. on the approach, and being 14 ft. apart at their ends. As the inside guard rails now extend 24 ft. beyond them, however, the timbers cannot exercise their former function of guiding a train into position to take the inside guard rails.

PENNSYLVANIA LINES.—The standard construction is shown in Fig. 7. The ties are spaced 8 ins. apart, which is more than is advisable. They are boxed out ¾-in. over the girder or stringer flanges, and are fastened to the bridge by clip washers on the ends of the guard-timber bolts. The guard timbers are lock-dapped 1½ ins. over the ties. The inside guards are not less than 85 lbs. or 75 lbs. per yd. for track rails of 100 lbs. and 85 lbs. per yd. respectively. They are full spiked to every tie and have rail braces on every third tie. At the entering end of the bridge they are extended 200 ft., brought together on a curve of 68 ft. radius for 14 ft. and bolted to a cast steel point. At the leaving end (on double track) they are extended 30 ft. and bent inward. The same arrangement is used on stone arch bridges and ballasted steel bridges where curves, great height, or other conditions make the protection desirable. All guard rail joints have the nuts on the inner side.

For curves on bridges having steel stringers 6½ ft. apart, the superelevation (up to 6 ins.) is given by using tapered ties, the thickness under the low rail being not less than 6½ ins. Where wooden floor beams are used, as on through plate-girder bridges, the elevation is given by blocks between the floor beam and outer rail. Ordinarily, the guard timber bolt serves to hold the deck to the bridge, as noted; but when the guard timber comes over the girder it is fastened to the ties by long screws and a clip bolt is used at every fourth tie.

SOUTHERN PACIFIC RY.—The ties are gained or boxed over the stringers so that there is no camber in the floor without live load on the bridge. The inside guard rails are 8 ins. from the track rails (3 ins. when rerailers are used); in addition to the spikes they have rail braces at every third tie on the bridge, at alternate ties for the straight part of the extension, and at every tie for the curved part of the extension. Where very light rails are used for the guards, they are supported by combined chairs and separators placed on every sixth tie on the bridge and every fourth tie on the approaches. The rerailers are described below.

The guard timbers are in 25-ft. lengths, scarfed vertically instead of horizontally at the joint, and have two horizontal bolts through each joint. They are secured by hook bolts in every fourth tie; the hooks engaging the outer flanges of stringers or girders. The ties are also fastened by hook bolts which engage the inner flanges. All nuts are set up tight and bolts checked to prevent any loosening. The ties which have the guard timber bolts are boxed 1½ ins. on top to receive the guard timber. At all other ties the timber is boxed 1½-in. to fit the ties and to serve as a spacer. Where creosoted ties are boxed or cut, the exposed wood must be given a coat of creosote paint. A 2 × 12-in. plank is laid along the center of the track, and is secured to the ties by 40-penny nails. The actual weight of the deck is about 405 lbs. per foot of single track, but a weight of 500 lbs. is assumed in estimating.

WABASH RY.—On wooden bridges, every third tie is fastened to the stringers by lag screws ¾ × 10 ins. On steel bridges the ties are boxed over the girder flanges, the depth of cut being 1 in., or not exceeding 2 ins. for girders having cover plates. They are also secured by the ¾-in. guard timber hook bolts in every third tie. The holes for these bolts are bored by augers not larger than ⅞-in. On deck girders and stringers of through trusses with 6½ ft. spacing, and for deck trusses and girders of 8-ft. spacing, the hook bolts engage the outer flanges. For deck trusses of 10-ft. spacing the bolts engage the inner flanges of the chords. The ends of the inside guard rails, where brought together on the approach, are bolted to a special point. The guard rails and track rails are spiked to every tie on the bridge and approach, which is better practice than to spike guard rails only to every second or third tie. A special feature in the bridge practice of this road is that no beveled ties are required for the deck construction on curves, the superelevation for curves being provided in the substructure or superstructure.

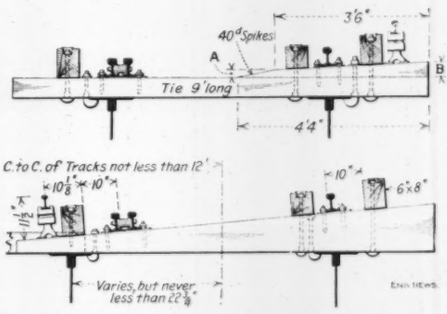

Fig. 13. Deck and Guard Rail Construction for Curves; South Side Elevated Ry., Chicago.

Rerailing Devices at Bridges.

A device which presents direct advantages and which should be used more extensively is a rerailer which will replace derailed wheels upon the rails before they enter the bridge, but devices of this kind appear to be almost obsolete. Of the several railways which have furnished us with particulars of their practice, the Boston & Maine Ry. and the Southern Pacific Ry. alone use rerailers. The Atchison, Topeka & Santa Fe Ry. has some at truss bridges built by the

TABLE III.—DECK AND GUARD RAIL ARRANGEMENTS ON OPEN-FLOOR ELEVATED RAILWAYS.

	Chicago South Side	Boston
TIES, Size	6 × 8 ins. × 8 ft.	7 × 8 ins. × 8¼ ft.
Spacing, c. to c.	18 ins.	12 ins.
Fastenings	Hook bolt in alt. ties (A)	Hook bolts in each tie.
OUTSIDE GUARD TIMBER (on edge)	6 × 8 ins.	6 × 9 ins.
Distance from gage of rail	11 ins.	10½ ins.
Fastenings	Bolt to alt. ties (B)	.....
INSIDE GUARD TIMBER	6 × 6 ins.	6 × 6 ins.
Distance from gage of rail	4 ins.	8 ins.
Fastenings	Bolted to alt. ties (A)	.....
INSIDE GUARD RAIL ON CURVES	.....	100-lb. rail.
Flangeway, 90 ft. curves	.....	2½ ins.
Flangeway, 500 ft. and over	.....	1¾ ins.

San Joaquin Valley Ry., but does not use them in its own practice. The New York, Ontario & Western Ry., which was for many years notable for its extensive use of the Latimer rerailer, reports that it no longer uses it. The Chicago, Burlington & Quincy Ry. has had considerable experience with the Latimer rerailing guard, and has found it successful, but its use has been abandoned for the reason that the inside guard rails are considered to prevent wrecks equally well, although they do not rerail the wheels. On this road, the rerailer is considered to have two defects: (1) the throat is too narrow to allow of its being connected with an inside guard rail so that a truck dropping off the rail on a bridge might cause a wreck; (2) there is liability of the guard being choked by ice and snow. As to the first objection, it is to be noted that the Southern Pacific Ry. uses inside guard rails with its rerailers; while as to the second, it does not seem that there should be more trouble than at frogs and switches.

The rerailing device of the Southern Pacific Ry. is placed about 30 to 50 ft. from the bridge, at which point the guard rails begin to flare inwards. Castings are fitted between the track and guard rails and outside of the former. These are so shaped as to guide derailed wheels towards the rails, and to carry the flange of the outer wheel across its rail so that it will drop into position. Where this device is used, the distance between the track and guard rails is only 3 ins., and the two rails are bolted together, with separators fitted between the webs. The general arrangement is shown in Fig. 8, and the details are shown in Fig. 9. The former shows also the arrangement with guard rails of old 50-lb. rail; they are supported by chairs which raise them to the proper level and which also form the separators. These castings are placed on every fourth tie on the approach, and every sixth tie on the bridge. The Boston & Maine Ry. uses rerailing guards at the approaches to some "jack-knife" drawbridges, and this arrangement is shown in Fig. 10.

Guards on Solid Floor Bridges.

Guard rails are not used as a general rule on bridges of this type, whether of steel or masonry, but it is desirable that such protection should be provided. Outside or inside guards are used in some cases where a narrow deck, great height, sharp curves, or other conditions render them specially desirable. For unballasted bridges with steel decks, sometimes used in track elevation work, the rails may be laid in a steel channel (of rolled section) or in a shallow trough composed of guard angles riveted to the deck.

A defective feature in concrete bridge construction appears to be the finishing of the coping below the rail level, with the ballast often sloped downward to the coping, so that a derailed car would have very little margin for safety. One reason for this is to facilitate tie renewals, the ties being slipped out endways over the coping without disturbing the track. But where this is done, the track should certainly be provided with inside guard rails. In Europe, masonry bridges have usually high parapet walls. This plan has been adopted on the masonry approaches of the Thebes cantilever bridge over the Mississippi (Engineering News, May 11, 1905).

In Fig. 11 is shown the arrangement of the track on the deck and through spans of a concrete truss bridge in France (Engineering News, April 22, 1909). In both cases the track (meter gage) lies in a trough 7 ft. 2 ins. wide, the sides rising several inches above the tops of the rails.

Elevated Railways.

On elevated railways, which are continuous viaducts, the almost universal practice is to provide guard timbers on both sides of each rail, substituting steel guard rails on curves. It is to be noted that the inside guard rails are higher than the track rails, but on ordinary railways it is generally considered inadvisable to allow the guards to extend above the level of the rail heads. The arrangements on the Chicago South Side Elevated Ry. and the Boston Elevated Ry. are given in Table III., and the construction for tangents and curves on the former road is shown in Figs. 12 and 13.

On the Chicago line, the curve superelevation is from ⅞-in. to 4½ ins. For the sharper curves this is given by elevation blocks on top of the ties, secured by the track spikes, guard rail bolts and hook bolts. For curves of over 800 ft. radius beveled ties are used, as shown. On curves, also, the inner guard timber is replaced by a steel guard rail bolted to the track rail, the width of throat or flangeway varying as noted in Table IV.

Bridge Guards on English Railways.

In Europe, it is very general practice to provide heavy outside guards, extending above the rails and well braced from the floor system. Some of the arrangements used for steel viaducts on railways in England and Scotland are shown in Fig. 14. These were given by Mr. Herbert T. Procter in a paper on track construction for viaducts, presented to the Institution of Civil Engineers (London) in 1892. In regard to the deck construction, longitudinal timbers laid upon the floor system have been used to carry the rails, but Mr. Procter preferred a solid floor with the usual arrangement of ballast and ties. This is cheaper and more durable but involves greater dead load and consequently a heavier structure. This can be reduced to some extent by limiting the width and depth of the ballast bed, and by using ballast of light weight (cinders or slag on a base of coarse slag). All the arrangement shown (except that of the Forth Bridge) provide for outside guards only, although inside guards are really the most effective and most valuable, as already shown. It is stated, however, that outside guard rails close to the track rails (but slightly above them) have been used on many small viaducts. Thus, on the old Tay Bridge, the guard rail (double-head section) was 2 ins. outside of and 1 in. above the track rail, both being carried in the same chairs. In any chair track, derailed wheels are likely to smash and tear up the track. It is better to so arrange the track and deck as to give a smooth riding surface for derailed wheels and to provide it with efficient guards to guide them across the bridge.

In regard to these arrangements shown in Fig. 1, (A) is that used on the Forth Bridge. Here each rail is in a deep steel trough, and derailed wheels would run on a continuous timber bearing. The track on the Britannia tubular bridge (B) was laid originally without any guards, and the two side timbers and guard angles were added later. The main longitudinal timber is slotted to fit over the web plate to which the angle brackets are riveted. Mr. Procter stated that: "On both of these bridges there is evidence of the guard having been effectual." The Montrose bridge (C) has a longitudinal timber for each rail, held in position by two angles forming a trough. The outside guard timber is bolted to the floor and to side braces; it is faced with a steel angle which is 7½ ins. from and 2½ ins. above the rail head.

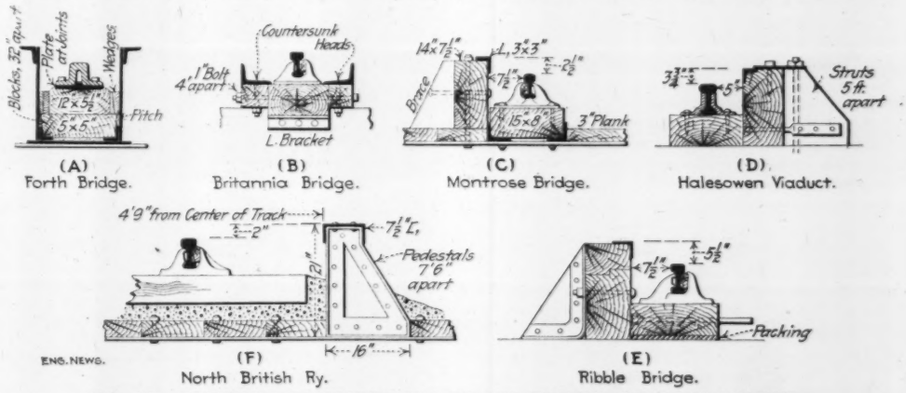

FIG. 14. EXAMPLES OF BRIDGE GUARDS ON ENGLISH RAILWAY BRIDGES.

The Halesowen Viaduct (D) has the rails supported by bars resembling continuous angle splice bars, and there are outside guard timbers 5 ins. from and 3¾ ins. above the track rails. The spacing must of course be outside the clearance limits of the rolling stock in any case. The Ribble Bridge (E) has a somewhat similar arrangement. The standard design of the North British Ry. is shown at (F). The ordinary ties are laid in ballast on a plank flooring, and steel pedestals (7 ft. 6 ins. apart) are riveted to the floor beams to carry a steel channel which forms a guard rail 2 ins. above the level of the track rail. On the new Tay Bridge, the guards (outside) consist of heavy timber conduits, one for a 9-in. water main and the other for signal and telegraph wires. They are 28⅝ ins. from the outside of the rails, and extend about 8 ins. above them. They form walks for the section men and bridge watchmen.

A steel deck construction with side guards, on the same general principle as the concrete construction shown in Fig. 11, has been used in some recent single-track viaducts on the Wemyss Bay Ry. The plate girders are 9 ft. 8 ins. apart, and carry a solid deck of transverse troughs 12

TABLE IV.—CURVE ARRANGEMENTS FOR FLOOR OF SOUTH SIDE ELEVATED RY.; CHICAGO.

Curve radius, ft.	Throat at inside guard, ins.	Width of gage, ft. ins.	Super-elevation, ins.	Thickness of ends of 9½-ft. bevel ties. Small, ins.	Large, ins.	Thickness of ends of elevation blocks. Small (A), ins.	Large (B), ins.
Over 2,000	1¾	4 8½	⅞	5⅝	7¼	⅝	15/16
1,201 to 2,000	1¾	4 8½	1⅞	5¼	8¾	15/16	21/32
801 to 1,200	1¾	4 8½	3	4⅜	10⅝	2⅛	4¼
Up to 800	1¾	4 8½	4½	4	12⅝	3⅜	6⅜
			For Sharp Curves.				
Radius, ft.		96 to 123	124 to 172	173 to 287		288 to 864	Over 864
Gage		4 ft. 9 ins.	4 ft. 8⅞ ins.	4 ft. 8¾ ins.		4 ft. 8⅝ ins.	4 ft. 8½ ins.
Width of throat at inside guard		2¼ ins.	2⅛ ins.	2 ins.		1⅞ ins.	1¾ ins.

ins. deep. On this deck are vertical plates (with an angle top and bottom) retaining the ballast. These are 9 ft. 8 ins. apart and about 22 ins. high, or 2 ins. above the tops of the double-head rails. The ballast plates and brackets on the ends of the deck carry concrete sidewalks about 2 ft. 9 ins. wide, the distance between hand-rails being 15 ft. 2 ins. It would be a difficult matter for a derailed train to burst this sidewalk and its steel framing, while in order to ride over it the wheels (riding on the ties) would have to climb a height of about 8 ins. Nevertheless, it would seem that inside guard rails close to the track rails would be advisable for guiding the train across the bridge, as the outside guards are about 28 ins. from the rails.

Hammer Drills Applied to Rock Excavation in Sewer Construction at Bloomington, Ind.

By GEO. L. THON.*

The construction of a sewerage system for the City of Bloomington, Ind., although urgently needed for years, was an undertaking the successive city administrations kept putting off on account of the large amount of rock which would necessarily have to be excavated in order to construct such a system. At last, however, it was seen that this improvement was essential to the welfare of the city, and could no longer be delayed on account of the difficulty due to the rock. The City Council, therefore, in 1905, employed engineers to prepare plans and specifications for a sewerage system covering the principal business and residence portion of the city. This system consists of about eleven miles of sewers, varying in size from 8 to 24 ins. in diameter, with a complete sewage purification plant, all at an estimated cost of $102,000. The cost of the rock excavation to install this sewer system was estimated at about 20% of the total cost, and as is good practice in municipal work of this

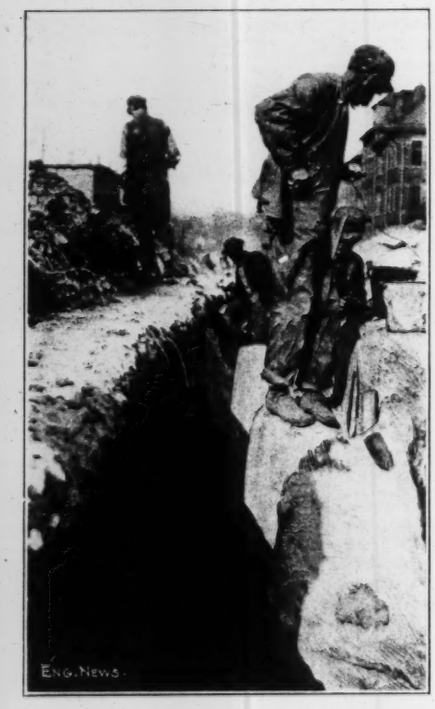

Fig. 1. Outcrop of Limestone in Trench at Bloomington, Ind.

character, was purposely estimated liberally in order that the actual cost would not exceed the estimated cost.

Bloomington is located in Monroe County, in what is known as the oolitic limestone region. This stone underlies the entire city and in most places is found very close to the surface of the ground. The oolitic stone is the commonly-called Bedford limestone which is quarried extensively in this locality for building purposes. The stone is found close to the surface of the ground, and in many cases outcrops in the surface of the street. Where it is covered with soil, the rock surface is generally very uneven as was subsequently shown when the trenches were opened. Sometimes there would be a series of "hogbacks" (Fig. 2) extending across the trench, which were from 2 to 8 ft. wide outcropping or nearly outcropping on the surface, and between these "hogbacks" would be seams of clay extending sometimes even below the grade line of the sewer. It is evident from the nature of such a rock contour that very little information could have been obtained, for the purpose of preliminary estimates, by putting down test holes, even though the holes were located comparatively close together. For instance, on Sixth St. on the north side of the Public Square, where the sewer was laid in a 10-ft. cut, the rock outcropped in several places, and yet at a distance of not 10 ft. from these outcrops along the line of the sewer, the pipe was laid without any rock excavation. For the purpose of making the preliminary estimates, the excavations made previously for the laying of gas and water mains were largely depended on for estimating the rock.

The average depth of the sewers was about 6½ ft., except about 1,000 ft., which was laid in a cut of from 9 to 12 ft. The rock excavated from the trenches varied from a few inches to 12 ft. in depth. The clay overlying the rock was of the stiff red variety found in this part of the state. This clay is very hard when dry, but on becoming wet becomes soft and caves easily, especially when jarred by the blasting of the rock underneath.

The rock excavation being a large item in this contract, the necessity of an economical method of rock excavation was apparent. The contractor's attention was called to the use of hammer drills for this kind of work, and after obtaining one of these drills and testing its use on work of this kind, and being satisfied with the results obtained, installed two air compressor outfits with six hammer drills for each outfit.

OUTFIT.—One of the air compressors installed was steam driven and the other was belt driven. The belt-driven machine was a 10 × 12-in. single stage air compressor, rated at 140 cu. ft. of free air per minute at 100 lbs. pressure when operated at a rate of 150 r.p.m. The other was a steam-driven single-stage 8 × 9 × 12-in. air compressor and rated at 124 cu. ft. of free air per minute at a pressure of 100 lbs. at 150 r.p.m. Both compressors were driven by a 16-HP. traction engine.

Each compressor outfit included about 1,000 ft. of 1½-in. wrought iron air pipe, 6 hammer rock drills with the necessary steel bits, 7-16-in. air hose, reheaters, valves, etc., with a small 7 × 9-ft. tool house. The tool house was mounted on a skid so that it could be pulled along the line of the work with the rest of the outfit by means of the traction engine. The belt-driven air compressor was also mounted on a skid, and the steam-driven compressor was mounted on a set of four heavy wagon wheels. The compressors were pulled along the line of the work as it progressed by the traction engines.

The cost of an outfit of this kind was as follows: The air compressors, each including an air drum, 1,000 ft. of 1½-in. pipe with valves, etc., cost about $800 each. The hammer drills cost $55 each. Fifty feet of 7-16-in. air hose for each drill cost $9, and the steel bits, which were of extra quality of steel, cost $2.50 each, for the three-foot lengths and proportionally more or less for the 2 and 4-ft. lengths. The total cost of each drilling outfit was about $1,300. The two traction engines that were used to drive the air compressors were rented at the rate of $5 per day from a local man. This included the cost of an engineer for each engine. The contractor furnished the fuel and oil.

These compressor outfits ran five drills successfully at a distance as high as 1,200 ft. from the air compressor, and by running the air compressor up to 110 lbs. pressure five drills could be operated at a distance of 1,500 ft. from the air compressor. On account of friction losses and leakage in the main air pipe, 1,500 ft. was about as far from the air compressor that the drills could be economically operated.

The outfit was generally moved at intervals of about 1,000 ft. along the line of the work, as it was easier to move the entire outfit than to operate with a longer air line. Moving the compressor outfit was a very simple matter. The pipe was unjointed and placed on a

Fig. 2. Hammer Drill Working on "Hogback" in Sewer Trench at Bloomington.

rack on the side of the tool house and then the air compressor and the tool house were chained to the rear of the traction engine and the whole outfit dragged to the next place where it was needed. In drilling close to the air compressor six drills could easily be supplied with sufficient air for drilling, but ordinarily only five drills were used, the extra drill being kept on hand for use in case any of the other drills had to be sent to the shop for repairs.

During the winter when the temperature was below freezing, heaters were used to reheat the air just before it entered the drill. These heaters were placed in the air line between the main air pipe and the drill. The heaters were made of a ½-in. pipe 16 ft. long, which was bent in the form of a coil so that it could be inserted in the cylinder of an ordinary kerosene oil heater. The air supplying the drill passed through this coil of pipe, which was kept heated by the oil heater. In passing through this heated coil of pipe it became warm and materially increased the efficiency of the drill in cold weather. Another form of heater used for the same purpose, and which proved even more satisfactory than the kerosene heater, was made by inserting a piece of 3-in. pipe 15 ins. long in the pipe supplying the drill. The 7-16-in. air supply pipe was enlarged by means of a bushing to the size of a 3-in. pipe, and then at the other end of the 3-in. pipe was reduced to the original size of the air supply pipe by the same means. This short enlarged section through which the air supplying the drill must pass at a decreased velocity is kept hot by means of a coal fire in a salamander. These methods of reheating the air proved fairly satisfactory and made it possible to work the drills on the coldest days during the winter.

The drills used are comparatively small, weighing about 20 lbs., and are easily handled by one man, a feature that led largely to their adop-

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tion in this work. The bits used in this work were of 2, 3 and 4-ft. lengths. When more than four feet of rock was excavated from the trench, 4-ft. holes were drilled and blasted and then the rock was redrilled. The bits are made of a hexangular steel shank 1 in. in diameter, with a 5-16-in. hole running from end to end through the center of the bit. The cutting end of the bit is dressed to 1¼-in. gage and finished with a "rose" or eight winged cutting end. Part of the exhaust air from the drill is discharged through the hole that extends down through the bit to the bottom of hole being drilled, and in this way the hole is kept clean from dust. The drill when the bits are newly sharpened bores a hole 1¼ ins. in diameter, but as the bits wear the size of the hole drilled decreases. When the hole drilled becomes less than one inch in diameter the bits are resharpened.

OPERATION.—But one man was required to operate a drill, and almost any ordinary laborer can learn to operate the drill in a few minutes and get good results. In fact, all the drillers on this work were selected from the laborers on the ditch and were men who had never seen a drill of this kind before. The men operating the drills were paid 17½ cts. an hour, which was but 2½ cts. an hour more than the wages paid for ordinary labor.

The cost of operating each drill outfit per day was approximately as follows:

Rent of traction engine, including engineer	$5.00
Five drillers, at 17.5 cts. per hour	8.75
Fuel and oil	3.00
Blacksmithing and new steel	1.00
Foreman	3.50
Water boy	.50
Total cost per day	$21.75

With this outfit in a dry trench and fairly uniform rock, 5 drillers drilled 263 holes, each hole 3 ft. deep in a shift of 10 hours, or a total of 789 ft. of hole at a cost of 2.75 cts. per lin. ft. of hole drilled. This is probably about the average cost of drilling under reasonably favorable conditions in a dry trench. In a wet trench where the rock surface was covered with mud and water, the speed of drilling was somewhat retarded. In one case, however, in a trench where the majority of holes were drilled in rock covered with from 2 to 4 ins. of water, one man drilled 40 holes each 3½ ft. deep in seven hours, which is at a rate of 0.33 ft. of hole per minute. In another instance one man drilled 160 holes, each 18 ins. deep, in 10 hours, or at a rate of 0.25 ft. of hole drilled per minute, this being also in a wet trench.

The drillers experienced considerable difficulty in places, due to mud seams, which often underlaid the rock surface. When the bit struck a mud seam it would generally choke up the exhaust hole of the bit. When this happened, the hole drilled in the rock would not be cleaned from dust as it went down and difficulty was then encountered in getting the bit out of the hole when the drilling was finished. In some cases it was necessary to leave the bit in the hole until the rock was broken up by the blast from the adjacent holes.

The accompanying table shows the average rate of drilling holes by means of these drills as used in this work under actual working conditions:

RATE OF DRILLING 1¼-IN. HOLES IN OOLITIC LIMESTONE WITH HAMMER DRILLS, BLOOMINGTON SEWERAGE SYSTEM.

Depth of hole drilled, ins.	Time started.	Time stopped.	Total time per hole, mins.	Depth of hole drilled per minute, ft.
16	11.44 a. m.	11.47 a. m	3.	.45
20	11.44 a. m.	11.47 a. m.	3.	.55
18	11.48.5a. m.	11.51 a. m.	2.5	.60
22	11.51.5a. m.	11.54 a. m.	2.5	.73
18	11.51 a. m.	11.54 a. m.	3.	.50
24	11.56 a. m.	12.00 a. m.	4.	.50
18	11.58 a. m.	12.00 a. m.	2.	.75
24	1.29 p. m.	1.31 p. m.	2.	1.00
24	1.24.5p. m.	1.27.5p. m.	3.	.66
24	1.28.5p. m.	1.32 p. m.	3.5	.57
3 consecutive holes each 4 ft. deep	1.30 p. m.	1.45 p. m.	15	.80

The data in the table were taken during the progress of the work; the drillers did not know that the time was being taken, and were therefore making no exceptional efforts to set records that would not be maintained in actual work. The table gives only the time occupied in drilling the hole, the time occupied in changing from one hole to another being included only in the last instance, where the time was taken for drilling three consecutive holes each four feet deep. While the table shows the rate at which holes can be drilled with these hammer drills, still for general information as to what can be accomplished with an outfit of this kind per day, the results given previously in this article would be a better indication as to what to expect from drills of this kind.

The distribution of holes put in the rock for the purpose of blasting varied with the nature and shape of the rock to be excavated. Where the rock was fairly uniform in depth and level on the surface, holes were placed on either side of the trench every 18 ins. measured along the ditch. A center hole was placed in the center of the trench, halfway between every alternate pair of side holes. Where there was less than one foot of rock to be excavated no side holes were drilled, the center holes alone being used to blast the rock. In some cases the center holes were omitted and the side holes were placed one foot apart. This arrangement of holes also gave good results in blasting. All holes were drilled 6 to 8 ins. below the grade of the sewer.

The holes were loaded with from 2 to 6 ounces of 60% dynamite. Four-ounce ⅞-in. sticks were used. This size stick of dynamite could easily be loaded in the holes drilled with the 1¼-in. drills. From 15 to 20 holes were loaded at once and connected into series with electric exploders and fired with a blasting machine. The blast broke the rock up so that it could be removed from the trench without further difficulty. The blasts were protected by laying a lattice made of 2-in. plank over that portion of the trench where the shot was to be fired, and a number of wooden poles were piled on the lattice, all of which was chained together with two log chains. Covering the blasts in this way proved a very effective way of protecting the blasts, and kept the broken rock from flying out of the trench.

The average cost of the rock excavation, including drilling, explosives, and removing the rock from the trench was $3.40 per cu. yd. In computing the rock yardage the trench was assumed to be 2-ft. wide.

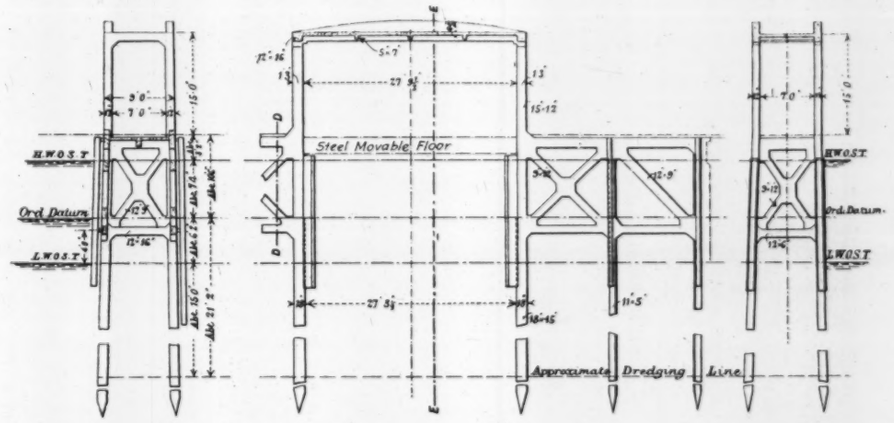

FIG. 1. DETAILS OF REINFORCED-CONCRETE FRAME-WORK FOR VERTICAL LIFT DRAWBRIDGE, HASLAR, ENG.

Fig. 2. View of Vertical Lift Drawbridge in the Haslar, Eng., Jetty.

The construction of the work was started Sept. 1, 1908, by the Independent Construction Co., of Terre Haute, Ind., with W. H. Harris, Manager of the company, in charge of the work on the ground. The system was built according to the plans and under the supervision of John W. Alvord and Chas. B. Burdick, of Chicago. These engineers were represented on the ground by the writer.

A Small Vertical-Lift Drawbridge with Reinforced-Concrete Frame.

A vertical-lift drawbridge with a reinforced-concrete framework has recently been built in connection with a new station for British submarine vessels at Haslar, Eng. The main construction at this port consists of two reinforced-concrete jetties enclosing the minor harbor in which the submarines are docked or anchored. These jetties, after the style of many now being built in Europe, are of the trussed type composed of a solid floor resting on two X-braced trusses, cross-braced horizontally, and resting in turn on piles driven in the sea bottom. Reinforced concrete is used for the entire construction.

In order to allow a sea opening inside the jetties as well as a continuous passage along the solid floor, it was necessary to put in a drawbridge. This is shown in the two accompanying figures. It is composed of two end vertical bents, which are merely continuations of the jetty bents and which carry at their top two longitudinal beams spanning the 27 ft. 9½ in. opening. These beams are 12 ins. wide and 15 ins. deep at the ends deepening to 33 ins. at the middle, and carry a beam and slab floor connecting them. All of this framework is of reinforced concrete, but the bridge proper, shown in Fig. 2, consists of two transverse solid web steel plate girders carrying two longitudinal steel lattice girders, with a solid floor. This is raised and lowered by chains and pulleys as shown in the half-tone.

The jetty itself is worthy of record. It is in two parts, one 855 ft. long and one 448 ft. long, made up for the whole length of the trussed system shown in the views. It is reinforced according to the Hennebique system. For the length of each part the main bearing girders are continuous over the bents, which are spaced some 14 ft. apart. A descriptive article of the complete work appeared in London "Engineering," June 11, 1909, from which this note is abstracted.

A Reinforced-Concrete Spillway with Concentrated Crest Length.

By D. W. MURPHY.*

The accompanying plans and illustrations show a spillway indented by a number of bays in such a way as to give an effective discharge length of more than three times the straight end-to-end measurement of the weir. The spillway, which was recently designed by the writer and constructed by the United States Reclamation Service on the Klamath Project, at Klamath Falls, Ore., is located on the Keno Canal, where it is proposed to divert 600 sec. ft. of water to be used for power purposes. The fall from the canal, at the point of diversion, to Link River, into which the water is discharged after being used for generating power, is approximately 48 ft., and the distance from the canal to the river is about 170 ft. (see Fig. 4).

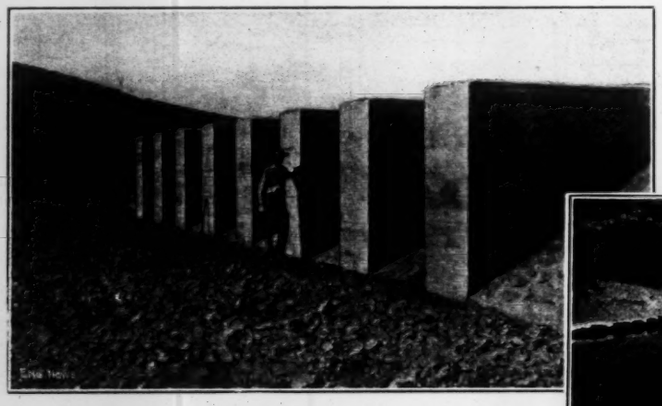

Fig. 1. Up-Stream Face of Overflow Walls During Construction.

Fig. 3. Down-Stream Face of Overflow Walls on April 1, Looking Up-Stream Toward the Crest.

FIGS. 1, 2 AND 3. REINFORCED-CONCRETE SPILLWAY ON THE KENO CANAL, KLAMATH PROJECT, U. S. RECLAMATION SERVICE, NEAR KLAMATH FALLS, OREGON.

The length of the canal, from its intake at the head of the river to the power site at the lower end of the rapids, is approximately one mile. The canal for this distance is along the side of a comparatively steep slope, where conditions made impracticable the constructing of banks of sufficient height to provide for any considerable rise of water in the canal above the normal flow-line. To meet these conditions, a spillway was required which would take care of the waste water due to regulation at the power-house without causing excessive rises in the canal. The amount of rise in the canal which was considered allowable was taken as 1 ft. In order to provide for an emergency, such as the sudden closing down of the entire power plant, a spillway was required with a capacity equal to the total amount of water to be diverted. The length of overflow weir necessary for this quantity, with the head available, using a safe coefficient of discharge for a round-crested weir, is approximately 200 ft.

The principal problem involved in the design was to provide a channel sufficient to collect the overflow from a long weir and conduct it over an earthen slope to the river below. To fulfill these conditions required a channel of comparatively large width and cross sectional area, in its upper portions, and also one capable of withstanding high velocities. To simplify the question of collecting the water and reduce the width of the upper portions of the channel, the weir was so constructed as to give the required length of overflow and yet discharge over a comparatively narrow width. This was accomplished, as shown in Fig. 4, by making the weir not straight but indented, in plan, by seven bays, into which water from the canal was free to flow. Thus, when the level in the canal rose, the water overflowed the entire crest, including that of the side and end walls of the bays. The length of the bays was made 10 ft., with a width of 5 ft. at the upper, or canal, ends and with 3-ft. spaces between bays. This gives an effective length of crest of about 200 ft., with a total width of channel of only 61 ft.

The upper bays, or those on the canal side of the weir crest were designed with cross sectional areas varying along their length in approximate proportion to the quantities of water to be carried in the different parts of each bay (see Figs. 1 and 5), in order to provide for a nearly uniform velocity of approach to the different parts of the weir. This also allowed the floor of the bay to be raised at its lower end, and served to decrease the pressure on the walls. Tests made upon a weir of this type showed that the quantity discharged was practically the same as for a straight weir of equal length operated under the same head.

The main structure, consisting of the overflow and wing walls, rests upon a rock foundation, a continuous wall along the lower face of the structure being carried up from the rock to the floor of the channel. The upper bays were excavated to rock, and the space below the floor of each carefully filled with puddled material. In order to prevent cracks developing in the angles of the walls at the downstream end of the bays, the horizontal reinforcing rods were made continuous around the two sides and lower end.

The walls and floor of the channel below the downstream face of the main structure were founded upon earth. In order to allow for settlement in this portion without a tendency to produce stresses on the overflow walls it was not tied to them. Pyramidal blocks of concrete, resting upon the floor, were placed at the lower end of the bays to take the shock of the overflow where the fall is comparatively great. The floor of the channel consists of concrete slabs 12 ft. square with expansion joints between them. Ribs at each of the joints serve to reinforce the slabs, and also to hold the floor in position on the slope. The lower end of the channel is protected by a concrete wall carried down to rock foundation, a detail not shown on the plans.

The spillway was completed during the fall of 1908 and carried a partial head of water against it during the winter of 1908-9. During this period, however, the canal carried only a small quantity of water and the head was not allowed to rise sufficiently to overflow the spillway, which was not put in operation until about April 1, 1909.

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A Comparison of Contact Beds and Percolating Filters.*

By JOHN E. FARMER.†

It is difficult to give an accurate comparative cost of construction [of contact beds and percolating sewage filters], as there are so many details affecting the cost of both methods. In this paper I have confined myself to results only, these having been obtained by the treatment of the same sewage, but not concurrently, by both methods.

The principles of the methods are identical, the application and the use of them being the difference. The principles, briefly, are (1) exposing the liquid to be treated to a large surface area covered with living purifying agencies; (2) keeping a vast number of bacteria in prime working condition.

In the contact method, the liquid is presented to the bacteria "in bulk," all the air between the particles composing the bed being replaced by the liquid. The bacteria are allowed a period in which to act on the liquid, which is then removed, carrying away with it the products of the bacteria. The next operation consists in giving the bacteria an air supply, by allowing a period of air contact.

In the percolating method the first principle is effected by allowing the liquid to run in a fine stream over the particles covered with the bacteria, but not filling the air space, the remaining space acting as a continuous "air contact," and so allowing the process to go on for days together.

The feeding of the bacteria and the supply of oxygen proceed simultaneously, and are not separate operations as in the contact method. The unit taken for the contact method is quantity treated per cubic yard of material comprising the bed, the quantity that can be

*Paper read at a meeting of the Association of Managers of Sewage Disposal Works, June, 1909. Reprinted from "The Surveyor and Municipal and County Engineer" [London], June 18, 1909.

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TABLE I.—CHEMICAL RESULTS OBTAINED BY TREATING SEWAGE ON PERCOLATING FILTERS 4 FT. AND 7 FT. DEEP.

	Ammonia, parts per million.			Grains per gal.	
	Free.	Albuminoid.	Oxygen absorbed, 4 hours.	Chlorine.	Ammonias, nitrites and nitrates.
Tanks effluent	60.88	5.01	2.50	6.2	....
4 ft. half " .	1.15	0.52	0.30	6.2	3.010
7 ft. half " .	1.77	0.76	0.46	6.0	3.010

TABLE II.—COMPARATIVE QUANTITIES TREATED AND PURIFICATION EFFECTED BY SEWAGE CONTACT BEDS AND PERCOLATING FILTERS.

	Contact beds (4 years). Gallons.	Percolating filters (2 years). Gallons.
Total volume treated....	892,228,000	962,496,000
Average volume treated each sq. yd. per day..	61	173
Average volume treated each cu. yd. per day..	46	104
	Average reduction on sewage. Contact beds (4 years). Per cent.	Percolating filters (2 years). Per cent.
Oxygen absorbed, 4 hrs.	47.3	76.8
Albuminoid ammonia ..	40.6	73.8

treated depending upon the available water volume left between the particles, which is found in practice, after periods of working, to decrease in some cases to a great amount.

With the percolating system the same unit is used by many, including the Local Government Board. This would make a true comparison if the modes of presenting the liquid to the purifying agencies were identical, and the volume of liquid that could be treated by percolating filters depended upon the air space between the particles; but as the liquid has to run in a stream over them the quantity that can be treated is independent of this space.

The best unit to use for percolating filters is the volume treated per square yard or per acre in 24 hours, especially as the depth or the cubic capacity is no criterion as to the volume which can be treated. In proof of this I give the following results of a filter constructed in two halves, and fed by a revolving sprinkler, each half having exactly the same quantities to deal with. One half is 7 ft. deep, composed of 5 ft. of gravel and 2 ft. of clinker, the other half being 4 ft. deep, and composed of 6 ins. of gravel and 3½ ft. of clinker.

The average results of 21 samples of each half taken during a period of eleven months are given on p. 278. [Table I.]

It will be seen that although one-half has 75% more depth, it does not give as good a result as the other half; and taking it on the cubic yard unit it should treat 75% more liquid than the lesser depth, or, in other words, the 4-ft. side does in proportion to the cubic yard unit 75% more work with better results.

The figures of Table II. give the comparison between "contact" and "percolating," as regards volume treated on the same gross superficial area, volume per cubic yard and the results obtained. The figures for "contact" are the total quantities dealt with and the average results for four years, those for percolating filters being for two years, so that comparison may be on the volume treated rather than on the period of working. The percolating filters were constructed on the same site as the contact beds, by converting each contact bed into four circular percolating filters, which accounts for the less net available area of the latter. The net available area of each method was: "Contact," 10,019 sq. yds., or 13.258 cu. yds.; "percolating," 7,599 sq. yds., or 12.664 cu. yds. The contact beds were 4 ft. deep and the percolating filters 5 ft. deep.

During the four years the contact beds were working they lost in water capacity 62%, and at the end there was no reserve whatever, and only 68% of the sewage for which they were originally constructed could be treated.

In the case of the percolating filters, as will be seen from the above figures [Table II.] they have treated during two years a greater total volume, and a greater volume per square yard and per cubic yard, this being done on a smaller total available area, and with more than one-eighth of the area resting at a time. The volume treated per day after two years' working is the same as at the commencement. The labor required for percolating filters taken on the volume treated is about one-half of that required for contact beds, the cleaning of the material not being taken into account.

One method may be better than the other to adopt according to circumstances, but many details must be considered before one can say definitely which will give the best results at the least expense.

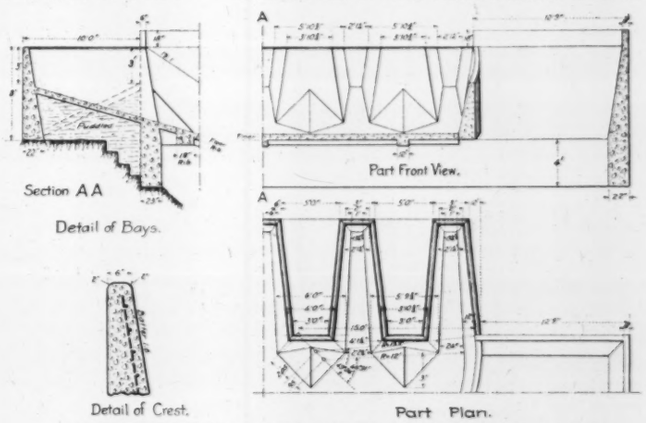
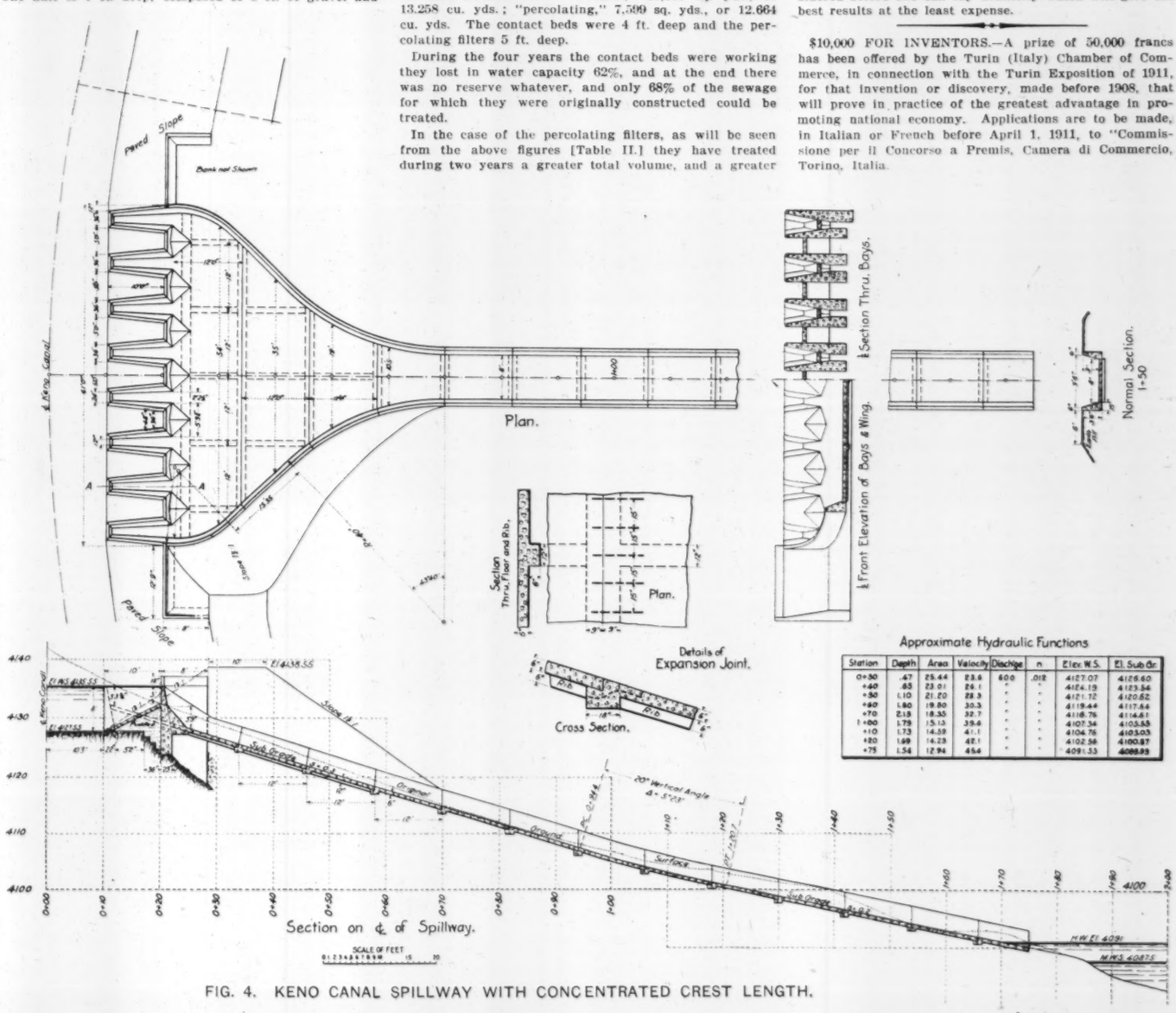


FIG. 5. DETAILS OF BAYS OF KENO CANAL SPILLWAY.

$10,000 FOR INVENTORS.—A prize of 50,000 francs has been offered by the Turin (Italy) Chamber of Commerce, in connection with the Turin Exposition of 1911, for that invention or discovery, made before 1908, that will prove in practice of the greatest advantage in promoting national economy. Applications are to be made, in Italian or French before April 1, 1911, to "Commissione per il Concorso a Premis, Camera di Commercio, Torino, Italia.



Station	Depth	Area	Velocity	Discharge	n	Elev. W.S.	El. Sub Gr.
0+30	.47	25.44	23.6	600	.012	4127.07	4126.60
+40	.83	23.01	26.1	"	"	4124.19	4123.36
+50	1.10	21.20	28.3	"	"	4121.72	4120.62
+60	1.80	19.80	30.3	"	"	4119.44	4117.64
+70	2.18	18.35	32.7	"	"	4116.76	4114.61
1+00	1.79	15.13	39.6	"	"	4107.34	4105.55
+10	1.73	14.59	41.1	"	"	4104.76	4103.03
+20	1.69	14.23	42.1	"	"	4102.56	4100.87
+75	1.54	12.94	46.4	"	"	4091.53	4089.99

FIG. 4. KENO CANAL SPILLWAY WITH CONCENTRATED CREST LENGTH.

The Purchase of Coal by the City of New York.

A recommendation for the purchase of coal, by the various departments of the City of New York, on a basis of the number of heat units contained, has been made by the Commissioners of Accounts in a recent report to the Mayor. This system of purchase and payment is needed to protect the city from the material losses suffered in the past, in the annual purchase of a half million tons of coal by accepting inferior grades through careless and dishonest inspection and by the failure of the purchasing officials to comprehend what constituted the true relative values of different coals.

The report of the Commissioner of Accounts was accompanied by a report of their chief engineer, Mr. Otto H. Klein, which briefly reviewed the previous attempts to establish a logical basis of coal purchase and described an investigation into the present practice of the several departments and boroughs of the city.

In general, it was found that all specifications in force were at variance with engineering principles, and that most of them were worthless in protecting the interests of the city. The only exception to this sweeping criticism was in the case of the Department of Bellevue and Allied Hospitals. The specifications in this one case prescribed narrow limits of size for the various grades and limited the ash content to 10%, demanding a content of 80% fixed carbon. These particular specifications were the only ones containing a requirement for purchase by heating value and the clause had never been enforced. This clause was based on a report of Mr. Klein, made in March, 1906, when the then Commission of Accounts recommended a revision of the practice of coal purchase.

The specifications of the Police Department were vague and prescribed only screening limits and the purchase of merchantable grades bearing well-known trade names as "Scranton," "Lackawanna," "Pittston," "Lehigh," etc. The Police Commissioner, late in 1908 and early in 1909, attempted to secure specifications in accord with the best engineering practice, but the Corporation Counsel ruled that the insertion of the heat-unit clause, was providing for the admission of an inferior article under a system of rebating that would lead only to constant disputing with the coal contractors to the loss of the city. Concerning this peculiar attitude of the Acting Corporation Counsel, Mr. John L. O'Brien, a report by Mr. Linus Seeley, Building Expert of the Police Department, is quoted as follows:

> He obviously failed to see, that if the coal were paid for according to quality and not according to quantity, that it would be an incentive to bidders to furnish the better quality; for the dealer who would furnish the poor quality of coal, of low heating power, would be at a disadvantage, in that while he would have the extra expense of hauling and handling the larger quantity of poor material to fill the station-house bins he would get less money for it than the dealer who furnished the good material, because payment is made according to amount of heating power and not according to quantity.
>
> In one case he is paid according to the pounds furnished, in the other, according to the heat units furnished. There is about as much chance of getting mixed in the percentage allowance as in the other.
>
> So the proposed method of buying coal, on which so much time and sincere effort has been expended, with the hope of establishing a more satisfactory scheme of buying and paying for coal, was knocked out, and the old unsatisfactory plan had to be returned to and the contracts for the 1909 supply made accordingly.

Mr. Klein closes his report with an example illustrating the purchase of coal by weight and by thermal value. Parts of this are given in what follows.

One department alone, in 1905, contracted for 80,000 tons of anthracite coal. Assume that there were three bidders on this contract and that their prices were as follows.

A	$6.50 per ton
B	$6.10 per ton
C	$6.00 per ton

and that "C" in consideration of the fact that he was the lowest bidder, secured the contract.

Suppose that the 80,000 tons furnished by "C" were just sufficient to do the required heating in a satisfactory manner during the year. Further assume that the percentage of ash and the calorific or heat value of the coal proposed to be furnished by each of the bidders were as follows:

	B. T. U. per lb.	Per cent. ash.
A's coal	13,000	12
B's coal	11,000	18
C's coal	10,500	24

It is obvious that A's coal has a greater heating power than either B's or C's, but C's coal costs less per ton than A's or B's. It is necessary to reduce all figures to a comparable basis. Now if 80,000 tons of C's coal will just do the work it is clear that it would require of B's coal for the same amount of heat:

$$\frac{80{,}000 \times 10{,}500}{11{,}000} = 76{,}363 \text{ tons.}$$

and of A's coal:

$$\frac{80{,}000 \times 10{,}500}{13{,}000} = 64{,}615 \text{ tons.}$$

The following statement can be made of the cost of heating with the several coals.

Bidder.	Tons required.	Price.	Total cost.
A	64,615	$6.50	$419,998
B	76,363	6.10	465,814
C	80,000	6.00	480,000

From this the remarkable fact appears that with a proper understanding of the quality of the three coals offered, the most expensive coal can be made to do the work at a saving of $60,000 over the cheapest.

When the method of comparison of bids, detailed in the proposed specifications, is used, this condition is at once shown to the official of the department letting the contract. Thus:

For A's coal,

$$\frac{13{,}000 \times 2{,}240}{650 + 6} = 44{,}390 \text{ B. T. U. for 1 ct.}$$

for B's coal,

$$\frac{11{,}000 \times 2{,}240}{610 + 9} = 39{,}806 \text{ B. T. U. for 1 ct.}$$

for C's coal,

$$\frac{10{,}500 \times 2{,}240}{600 + 12} = 38{,}431 \text{ B. T. U. for 1 ct.}$$

The economy in the use of A's coal is obvious and under such specifications the contract would be awarded to A.

It may be asked "What is to prevent C submitting, with his bid of $6 per ton, a sample of coal equal in quality to A's coal, and then filling the delivery orders from his own coal which is inferior in quality?"

The payment therefore is on the following basis:

$$\frac{\text{B. T. U. per 1 ct., delivered coal}}{\text{B. T. U. per 1 ct., bidding sample}} \times \text{price per ton.}$$

Thus, on the above assumption, C would receive for his coal:

$$\frac{38{,}431 \times \$6.00}{44{,}390} = \$5.19\tfrac{1}{2} \text{ per ton}$$

for the cargo or car-load from which the sample was taken, in place of the $6 per ton bid.

As an appendix to Mr. Klein's report, complete specifications for the purchase of coal are given, applicable to all departments of the city. The main clauses relative to sampling, payment, etc., are reprinted below:

QUALITY.— . . . The quality of the coal must be uniform throughout the period of this contract. It shall not contain in any case less than 75% of combustible matter and not more than 18% of ash.

SAMPLE FURNISHED WITH BID.—A representative sample of not less than 5 lbs. of coal proposed to be delivered under this contract, enclosed in a suitable air-tight receptacle, and plainly marked with the name of the bidder, the date, and the mine from which it was taken, shall be submitted for examination and test with the bids. A portion of this sample shall be tested for ash and combustible matter as previously indicated. If this test be successfully passed then the remainder of the sample shall be finely pulverized and then submitted to a calorimeter test, using either the Mahler Bomb Calorimeter or the Parr Standard Calorimeter. The results of the test shall be submitted to the various bidders, if written request is made for same. The result of the calorific test, together with the price bid per ton, shall form the basis for comparison and acceptance of bids. Tests made of the various samples of coal furnished with the bids shall comprise the following determinations:

Ash per cent in coal as furnished.....
Moisture per cent in coal as furnished.....
British Thermal Units in coal as furnished.....
British Thermal Units in dry coal.....
British Thermal Units net for 1 ct.....
Allowance for ash .....

DETERMINATION OF BID PRICE.—By multiplying the number of British Thermal Units as found per pound in the sample of the commercial or moist coal, by 2,240, and dividing this product by the bid price per ton (expressed in cents) plus one-half of the percentage of ash found in the sample (expressed in cents) the amount of net B. T. U.'s for one cent is found, and this value will be the basis for comparison and acceptance of bids and award of contract, the award to be made to the bidder whose sample shows the greatest number of B. T. U.'s per cent.

The addition of one-half the percentage of ash found in the sample to the price bid per ton is made for the purpose of compensating for the handling and disposal of ashes.

SAMPLING.—The coal to be tested and analyzed after delivery shall be taken by sampling from each cargo, or, if delivered by car-loads, from each ten car-loads, or as often as may be deemed necessary by the (official of department), and the method of taking the sample shall be as follows, the contractor furnishing the necessary labor:

The sample shall be taken from the cars or boat as the case may be at the point of delivery, and in the presence of an inspector or other representative of the department duly designated for the purpose by the (official of department).

The sample taken will in no case be less than the total of 200 lbs. for pea or smaller coal and 500 lbs. for larger size coal, and will be taken from numerous portions of the cargo or car-load, in order that it will in every respect truly represent the quality of coal under consideration. This large sample will be divided in four equal parts. Opposite quarters will be thrown out and the remaining portion of all sizes above pea coal will be reduced by a crusher to a size not to exceed 1 in. in diameter. This sample is again divided in four parts and opposite quarters thrown out. This process of quartering and crushing will be continued as rapidly as possible until the final sample thus obtained shall weigh about 5 lbs. and the largest fragment of which shall not exceed ¼-in. in its largest dimension. This final sample will then be placed in a shipping can or jar, sealed air-tight and forwarded to the chemical laboratory.

If desired by the coal contractor, he may have a representative present when the sample is taken.

FINAL CHECK TEST.—In case a test is not satisfactory to the contractor he may have an analysis made of a portion of the identical sample of pulverized coal at a laboratory to be agreed upon between the contractor and the city, and if the results of the analysis made at this laboratory be 2% lower than the results of the test of the original sample submitted with the bid, then the contractor will pay the cost of the second analysis; but if the result shows a deviation of less than 2% from the results of the test of such original sample, then the city will pay for the second analysis. This check test, which shall be made by the Mechanical Engineering Laboratories of either Columbia University, Stevens' Institute or Cornell University, shall be final and govern the payment under this contract.

DETERMINATION OF PRICE TO BE PAID.—It is intended that the calorific value of the coal in all deliveries shall be equal to that of the sample submitted with the bids, but any variance in heating value will be computed on the amount of the British Thermal Units for one cent, as follows:

Multiply the British Thermal Units net for one cent of delivered coal by the price per ton, and divide this product by the British Thermal Units net for one cent, as determined by the test of the original sample submitted with the bid. The quotient then will represent the price per ton to be paid for the cargo or car-loads from which the sample was taken.

REJECTION OF COAL.—It is understood and agreed, however, that no delivery of coal shall be accepted whose calorific value shall be less than 11,500 B. T. U. per 1 lb. of commercial or moist coal, nor shall the (official of department) be bound to accept any delivery of coal whose calorific value is satisfactory if the coal does not conform to the requirements of these specifications in all other respects.

A DOUBLE-STAGE CABLE-INCLINE RAILWAY at Ventnor (in the Isle of Wight), England, is proposed by the municipal authorities, in order to give access to the beach below and the hills above the town. The lower stage will have a rise of about 300 ft., with two intermediate stops; its horizontal length will be 1,614 ft., with a maximum grade of 1 in 3.50. The upper stage will have a rise of 350 ft., with a horizontal length of 1,130 ft. and a maximum grade of 1 in 2.70. Each line has two cars operated by a cable, the line being single track with a turnout or passing place at the middle. The power station for both lines is in the middle, that is, at the level of the town, so that the cables for the upper section will have to pass around head sheaves at the summit. The track will be of meter gage, with rails bolted to ties consisting of steel angles embedded in concrete. Each car will weigh about 7 tons empty, and carry 40 passengers in four compartments. A 40-HP. electric motor will operate the lower section, while the upper section will have a 30-HP. motor driving two drums, owing to the necessary arrangement of the cables. This is one of the special features of the proposed line, as practically all power railways of this class have been built with the driving drums and machinery at the head of the incline.

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The North Pole of the earth has at last been reached. That is the news of the week which in the popular estimation doubtless far outranks all other events chronicled in the daily papers. One would go far back in time, to find record of an achievement in geographic exploration that has awakened any such amount of popular interest. And the interest has been greatly stimulated, of course, by the announcement that not one but two explorers have reached that goal for which arctic expeditions have been striving nearly four score years.

The day is long past when the reaching of the geographic pole had any great importance in adding to scientific knowledge; but as a sporting feat it is a victory of the highest order and every American feels pride that the honor of accomplishing this feat goes to the United States. If there were any regret felt in this country when the news was flashed over the wires of Dr. Cook's marvellous success, it was regret that the intrepid Peary had not been the first to accomplish the task toward which he had devoted the better part of his life. But there was left little reason for regret when the news came that Peary also had reached the Pole and had reached it and returned to send the news of his final success with no knowledge that an earlier explorer had preceded him. Under these circumstances the honor of the achievement is hardly dimmed by the news of Cook's prior visit.

Reference is hardly needed at this time to the newspaper discussion as to whether Dr. Cook's statement is to be accepted. As a reputable scientist and member of a learned profession, it would seem to deserve acceptance until evidence appears for doubting it, or at least until a detailed account of the journey over the polar sea to and from "the end of the earth" is made public by the explorer.

Meanwhile, it is certain that the international race to reach the pole has been won by the United States.

Most unexpected and gratifying news reaches us from California. It tells that in a single bound the best American performance in rock-tunnel driving has been more than doubled, Europe's best figures equaled and surpassed, and a new world's record established. The following telegram to this journal from the Assistant Chief Engineer of the Los Angeles Aqueduct gives the facts:

August progress Red Rock Tunnel, Los Angeles Aqueduct, 1,061.6 ft. one heading sandstone. Three shifts of 13 men. Excavation cost less than six dollars per foot. Sectional area, 75 sq. ft. As in the case of all our tunnels, work done directly by city, with bonus system. J. B. Lippincott.

Hitherto, American tunneling figures were so far behind European records that to equal or surpass these seemed a hopeless impossibility. We summarized this state of affairs, with full citation of figures, in our issue of April 2, 1908. While subsequently the American record was slightly bettered, at Chicago in limestone and at Los Angeles in granite, it still remained very far behind the Simplon record. To make matters worse, the gap was suddenly widened this year, new figures being set by the builders of the Loetschberg Tunnel, in the Swiss Alps. When, in February, they resumed work in the unlucky North heading, it was with such fresh vigor of attack that the star performance in the Simplon Tunnel, 685 ft., was quite overshadowed by monthly records of 800 to 1,000 ft. The latest and best record, made in July, as reported in our last issue, was 1,013 ft. This is now exceeded by nearly 50 ft. in the record which Mr. Lippincott announces.

Besides fixing a new world's record, the Red Rock Tunnel figure has an added distinction. The Loetschberg record of 1,013 ft. is about 50% better than the prior European record. But we in America have never had such models of rock tunnel work as the Simplon before our eyes to guide us. The best American figures reported hitherto were 449 ft. (Gunnison), 466 ft. (Elizabeth Lake), 468 ft. (Chicago water-works), and 476 ft. (Elizabeth Lake). The Red Rock record of 1,061.6 ft. is fully 123% ahead of the latest of these, a marvelous percentage of gain.

Full details as to the methods and machinery by which this marvellous rate of progress was made will be awaited with great interest. It is probable that the sandstone in which the heading was driven was unusually favorable for drilling and breaking; but even in the best imaginable rock, the remarkable advance must have required most perfect organization. Perhaps some of the publications which habitually condemn the carrying out of work directly by an engineer, will be willing now to record the fact that all the hustling and "teamwork" is not confined to contractors' camps.

An element of good sanitation is orderly conduct on the part of city employees engaged in sanitary work. While it may not be absolutely essential that this orderliness be continued outside of working hours and during those outings which are demanded for purposes of recreation and the solidarity of labor, yet it is desirable that the proprieties be observed even then. All this being so, the people of our national capital must have been greatly relieved and highly gratified on reading the following "Special Notice" in the advertising columns of the Washington "Star":

TO THE PUBLIC.—The Garbage Men's Relief Club wishes to announce to the public that the rumor that the fighting of last Sunday morning on the "River Queen" was our excursion is false. The Garbage Men's excursion left Washington last Saturday night at 9 o'clock, landing at Washington Park about 10 o'clock, and back to Washington at 11.50 o'clock, without a fight, and everybody went home happy and no hurry-up wagon to meet them.

Seriously, this body of municipal laborers deserves commendation for so valuing its reputation as to go to the trouble and expense of defending it against false reports.

When private interests awake to the fact that inefficient municipal government costs them good money, we may expect to see municipal reforms of various sorts make even greater strides than they have been doing in the past. The foregoing remark is suggested by the leading recommendation of the National Board of Fire Underwriters in one of its recent reports on water-supply and fire-protection of a city of some 50,000 population. The recommendation is as follows:

That the ordinances be amended to require suitable qualifications for all officers and members of the water, fire, building and electrical departments, and to make the terms of office indefinite, so that duly qualified incumbents may be removed only for cause.

Presumably the committee in some other of its many valuable reports has made suggestions along the same line, but we do not remember of any so forcibly put as this one.

We have looked for some time past to see an awakening of the life insurance companies to the need of reforms in municipal and state health-protective matters. Such action on the part of life insurance companies is perhaps not so clearly and strongly demanded as in the case of fire insurance companies, owing to possibly greater complications in the first case, but the need and ultimate value of such action is perhaps greater on the part of the life than the fire insurance companies.

Ever since coal became the great fuel for warming buildings and for developing power, the public has purchased the several varieties largely on the statements of dealers or because of the general reputation of the mines from which it came. If a lot of coal has proved unsatisfactory there has been no recompense for the extra trouble and expense caused by the troublesome supply. The purchaser usually has been advised to blame his furnace or his firemen.

Such a condition was probably unavoidable in the earlier days when there was indeed little reliable information of the composition and character of the available coals. But now, owing largely to the accumulated efforts of the U. S. Geological Survey, this ignorance has been effaced. Not only have the general characteristics of the coal in the principal deposits been disclosed, but also methods have been made commonly available to determine the individual and relative values of specific samples, quickly, easily and cheaply.

With these better conditions now prevailing, it would be rank extravagance, obviously, for a private individual to purchase a half million tons of coal annually, without knowing how much he would pay for the valuable parts of his coal or how much he would lose by excessive wastes. For an official of a private corporation to impose such loose dealings on his concern would call down upon him an unpleasant investigation as to the nature of his connection with the source of supply. What then shall be said of an administration of the greatest city of the country, the second city of the world, which buys this half million tons of coal every year and knows not what it gets or pays for, while public works are halted pending an investigation into the debt limit of the city?

In this case responsibility is divided. No one department or official purchases the whole supply. One department purchases 80,000 tons, another perhaps 100,000 tons, another 150,000 tons possibly. It is recorded, as noted in an article elsewhere in this issue, that a former Police Commissioner attempted to have a reasonable basis of purchase adopted, but the plan was obstructed by the Corporation Counsel's office under the strange opinion rendered that the plan provided for the admission of an inferior article and for a system of undesirable rebating. That such a view was absurd is easily demonstrated, and perhaps in no easier way than by showing the results of the Federal departments or of public-service corporations which pay for their fuel according to the number of heat units locked up in it. Students of municipal economy may be forgiven for imputing a general obstructionist policy and all sorts of unholy alliances to the legal advisors who frame such excuses to squelch a needed reform.

The adoption of a rational scheme for the purchase of the coal supply of the whole city of New York is supported by the Bureau of Municipal Research and advocated by the Commissioners of Accounts. It may therefore reasonably be hoped and expected that some time this change will be made for the elimination of opportunities for favoritism and dishonesty and for the benefits of an added economy in the purchase and use of the fuel needed in the many departments and boroughs of the great city of New York.

The Protection of Trains on Railway Bridges.

The old question of the protection of derailed trains on bridges, which is still brought to public attention from time to time by an accident, has not quite the same aspect to-day as in former years. The very light deck construction, with widely spaced and ill-secured ties, which was at one time unpleasantly characteristic of American railway bridges, has practically disappeared, and the economy and advantages of a substantial deck are now generally recognized. Guard rails of some kind are also nearly universal. Yet the subject is still as important as at any time, and the varying practice of different railways merits study and comparison.

The disaster on the Atlantic City line of the Pennsylvania Ry., in 1906, one of the most serious railway accidents of recent years, was due to insufficient protection on bridges. In this particular case an electric train was derailed at a drawbridge by misplaced defective (and unprotected) end-rail connections, burst through the outer guard timbers, and went off the structure into deep water, causing the loss of 57 lives. The primary cause of the wreck was the use of a glaringly defective device for connecting the track rails on the bridge and the abutment. The secondary cause lay in the very inefficient system of guard rails. But the latter cause, while secondary, is really more important in relation to bridge protection, since the number of bridges and structures to be provided with guard rails is very much greater than that of the drawbridges to be fitted with end-lift rail connections.

The causes and lessons of the Atlantic City disaster were discussed very thoroughly in Engineering News at the time of the accident. More recently we have had occasion to inquire into the practice of several railways regarding deck and guard-rail construction on bridges. The details of their practice, as given elsewhere in this issue, suggest that the teachings of this disastrous wreck, and of other accidents of a generally similar character, have not yet found full application.

The ordinary practice for open-deck bridges is to place (1) a guard timber outside of and at some distance from each track rail, and (2) a steel guard rail inside of and closer to each track rail. In many cases, however, the guards are defective in strength or position; while in other cases there has been a tendency to omit the inside guard rails, which are by far the better preventives of accident in the event of a derailment.

It has been explained many times why the inside guard rails are the more important: they hold the wheels in a narrow space close to the track rails, the wheels and trucks are prevented from slewing at an angle to the track—a most dangerous movement, and they guide the wheel by bearing against the back of the flange so that the pressure tends to straighten the truck. With outside guard timbers, which are always at a greater distance from the rails, the truck may slew to a considerable angle before the wheels engage the guard; they engage it then with the facing side of the tread, which position increases any tendency to burst or climb the guard.

It has been argued that the outside guard timbers alone form a sufficient protection, but certainly this is not the case as long as the arrangement of these timbers is that commonly adopted. Conditions might be different with substantial outside guards close to the rails and built as part of the bridge floor. This is an arrangement used abroad, and is described elsewhere in this issue. With the ordinary construction adopted in this country, the outside timbers should not be regarded as wheel guards, but simply as a means of holding the ties in place when subjected to the blows of derailed wheels. Several railways have taken this position. In fact, on many through bridges the spacing of the guard timbers and trusses is such that (were inside guard rails omitted) the body of a derailed car would strike the truss before the wheels engaged with the outside guard timbers.

To act as tie-spacers, the guard timbers are usually boxed out to fit the ties, but some roads use flat timbers and rely upon the fastenings which attach them to the ties. In exceptional cases, spacing blocks are fitted between the ties, thus absolutely preventing the bunching of ties which leaves wide open spaces to cause violent shocks to the wheels. With ties closely spaced and securely held, a derailed truck may ride in approximate safety, but the conditions might be improved by providing a smooth unbroken path for the wheels, as is done in some of the special guards and bridge floor systems.

It is evident that the liability of derailed trucks to slew increases with the width of the space in which the wheels are free to wander. Where outside guards only are used, this liability increases, of course, with the distance between the track rail and the guard timber, while the efficiency of this timber to check or guide the truck is decreased in proportion to the angle of slew when the wheels strike it.

Yet in a paper on track and floor construction for railway viaducts, read some years ago before the Institution of Civil Engineers (London) by Mr. H. T. Procter, the surprising suggestion was made that with a solid floor a derailed train is equally safe at any part of the floor, and that therefore the guard may be placed at the extreme edge of the floor (where it is not in the way) in order to prevent a train from going over the side. This might be true if the train would kindly hold a straight course nearly parallel with the track. Experience has shown that this is not the case, even with the old style of English cars having (instead of trucks) two or three rigid axles carried in bearings in the underframe. The sooner the wheels are caught and guided, the better for the safety of the train and the structure, whether the rolling-stock is carried by rigid axles or by trucks. As a matter of fact, Mr. Procter's views are not indorsed by English railway practice, which usually provides very efficient bridge guards.

The inside guard rails have the special advantage (as noted above) of limiting the lateral movement of derailed wheels, preventing the tendency of the trucks to slew, and guiding the derailed cars across the bridge. The fact cannot be emphasized too strongly that the further the cars can go laterally, the greater is the liability of the slewing of the trucks. In wandering to a distant outside guard, they may slew to such an extent that wheels strike the guard at an angle which will result in a violent shock, with consequent liability of their breaking or riding over the guard, or even causing the car to be overturned or thrown from its trucks.

It is necessary, of course, to extend the guards beyond the ends of the bridge, in order to catch any "wandering" derailed car and guide it into position to cross the bridge floor safely. The older practice was to use the outside guard timbers as the primary guides for this purpose, extending them 20 ft. or more, and flaring them approximately to the width of the roadbed, long ties on the approach being used to support the guards. The inside guard rails were then extended to a distance of only about 10 ft. from the bridge, and were inclined inward to meet at a point in the center of the track. In recent years, however, the practice of extending the outer guards beyond the bridge floor has become obsolete. On the other hand, it is now almost universal practice to extend the inside guard rails to a much greater distance than formerly (from 50 ft. to 200 ft.), and to rely on these to catch and guide derailed cars.

At one time it was a very general practice to put heavy posts or piles on the approaches, in line with the trusses (or at the ends of the extended guard timbers), so as to stop any car that was so far off the track that it would not be caught by the guards and would strike the bridge (if of the through type). This was done partly to prevent the destruction of the bridge (with the train), and partly because it was considered a less evil to smash the car on the approach than to let it reach a deck bridge with the probability of going into the stream. This practice is now obsolete.

The rerailing guard, which was extensively used at one time, and which was a most efficient safety device, appears to be almost obsolete. Even the New York, Ontario & Western Ry., which was conspicuous for its general use of the Latimer-Childs rerailer, has abandoned this device. This old, well-tried and proved rerailing guard was described and illustrated in our issue of Nov. 8, 1906, while its practical value was testified by letters from correspondents in our issues of Nov. 15 and 22, 1906. Two instances of the present use of rerailing devices are noted elsewhere in this issue. It seems strange that the use of devices of this class should have been abandoned, as their efficiency has been proved and is generally admitted. At the present time, however, it appears to be the general opinion that with a good deck and guard-rail construction a derailed train may be carried safely across a bridge, and may then be left to take care of itself. But the approach is likely to be a long embankment (perhaps on a curve), and is not likely to be provided with guards of any kind. Thus the derailed trucks are free to swerve or slew and the train may go over the bank with results little less serious than if it had gone off the bridge.

In what has been said thus far we have dealt only with ordinary practice. It is worth while considering, however, whether it might not be advisable, in some cases at least, to depart from this practice and to introduce some more efficient system having the guards built as a part of the structure. This may be specially desirable for bridges over which trains are run at high speed. The tendency to introduce solid floor and ballasted floor construction for small and large bridges on important main lines has a bearing on this matter, as it might be easy and inexpensive to work in a substantial guard-rail construction as part of the floor system.

Various methods of effecting the improvement will suggest themselves, depending in part upon the general character of design of the bridge. It may be effected preferably, perhaps, by carrying each rail or each track in a trough of such width and depth as to reduce to a minimum the lateral travel of derailed wheels and to preclude any possibility of the wheels bursting through or climbing over the guard. Such construction should include either a solid deck, or at least a continuous and smooth surface to carry the wheels. Some examples of foreign practice in this direction are shown elsewhere.

In a few cases it has been argued that where the ordinary type of ballasted track is carried across the bridge (whether of concrete or steel) there is little need of providing guard rails. We believe, however, that inside guard rails should be used on all bridges. We may go further and say that they might well be used more extensively at such points as high embankments, especially where long or sharp curves occur. They are very desirable also in tunnels, and under bridges which are supported by columns or bents near the tracks. The Baltimore & Ohio Ry. requires inside guard rails to be laid in tunnels, while the concrete roadbed construction (Bush system) in the new Bergen Hill tunnel of the Delaware, Lackawanna & Western Ry. (Eng. News, Aug. 19, 1909) includes outside guard timbers.

Special means of protection are required at swing bridges. Reference has been made already to a very serious accident which was caused by defective end-lifts on a closed drawbridge. All drawbridges require the same means of protection for derailed trains as are provided on fixed spans. They require also protection against accidents due to a train running upon a bridge which is either swung open or is not closed in proper position. This protection is mainly a question of signals on the approaches, and the interlocking of the signals with the bridge mechanism. The signals may be operated from the bridge, or from towers on the approaches, the latter arrangement being perhaps necessary where there is heavy railway and river traffic. In either case, there is a fixed series of con-

secutive operations, no one of which can be made until those preceding have been made.

In the case of control of all operations by the man in the tower on the bridge, he must first set the railway distant signals at the "stop" position (at the same time perhaps opening derails or setting automatic stops). This action automatically releases the bridge locks and end lifts and allows him to withdraw them; this movement in turn releases the operating machinery and permits him to swing the bridge. When the bridge is again closed, it may be locked automatically, but it must be impossible for the operator to lower the railway signals until the bridge is locked and the rails are properly connected. Where signalmen on the approaches control the bridge, they must both set their signals at "stop" before they can release the bridge locks, and these locks are not under the control of the bridge operator. The withdrawal of the locks then releases the swinging machinery, and until the bridge is again locked the signalmen cannot operate their respective signals. In some cases the navigation signals on the bridge are interlocked with the machinery, so that the bridge cannot be swung until these are properly displayed.

The location of switches on bridges is rare, and is undesirable in every case. Where conditions necessitate it, there should be ample protection of the switch itself (to ensure its proper position or clear warning of its defective position), and special protection for trains in the emergency of their being derailed by the switch. The necessity for such precautions is emphasized by the accident on the Fraser River Bridge, noted in our issue of Aug. 5, 1909.

In conclusion, it may be well to point out that the necessity for protection of derailed trains on bridges applies to small as well as large bridges, and that it applies as fully to the bridges of electric interurban railways as it does to those of steam railways. It is not unusual to see small or old bridges equipped with guards of a very ineffective character, even on roads where more modern or important structures are properly equipped. But every such ill-equipped bridge is a danger point in the path of a derailed train, and a derailment is as likely to happen at one bridge as at another. For this reason, the inspection, overhauling and improvement of protective appliances on all bridges may be a wise and precautionary measure.

This measure is likely to be even more necessary on some electric railways, as many bridges and trestles on lines of this class have been provided with very inadequate protection for derailed cars. The cars are apt to be run at very high speeds, and as there is somewhat lax discipline on some of these lines, neglect of "slow orders" must be reckoned with. A single car, also, may have a bounding or swinging motion which is more severe than in a car coupled in a train, and which may accentuate any tendency to or cause of derailment.

We repeat, therefore, that the engineers of steam and electric railways may well institute a careful inspection and improvement of their bridge deck and guard rail systems with a view to the provision against possible disasters following the derailment of trains.

LETTERS TO THE EDITOR.

The Elder Trautwine's Attitude Toward Engineering Theory.

Sir: I have read with much interest Mr. Llano's paper, in Engineering News of Aug. 19, on "Engineering Science and the 'Practical Man' in Engineering," especially (of course) his reference to my father's preface to The Civil Engineer's Pocket-Book.

As remarked by Mr. Llano, the sentences quoted by him from the preface show that my father was completely in accord with him in his devotion to theory, and equally strenuous with him in his criticism of the so-called "practical man." As to my father's quarrel with those injudicious theorists who buried theory's "most simple facts . . . out of sight under heaps [not of mathematics but] of mathematical rubbish" and who enveloped the elementary principles of engineering theory "in such a haze of mystery as to render pursuit hopeless to any but a confirmed mathematician," I submit that this was the quarrel of a lover, not that of a despiser, of theory.

In judging of the attitude of our predecessors toward the theorists we should bear in mind the conditions which then obtained, notably (1) the comparatively elementary character of most of the problems then submitted to the "civil" (stationary or out-door) engineer, and (2) the paucity of theoretical literature then offered him, and its lack of adaptation to his needs. Could my father have read some of our modern engineering textbooks, he would have been as enthusiastic in his praise of them as he was severe in his comments upon the grave-diggers with their "heaps of mathematical rubbish."

Referring to my father's statement that "*nearly* all the scientific *principles* which constitute the *foundation* of *civil* engineering are *susceptible of complete* and satisfactory *explanation* to any person who *really* possesses only so much elementary knowledge of arithmetic and natural philosophy as is *supposed* to be taught to boys of twelve or fourteen in our public schools," Mr. Llano says: "Every educated engineer knows that this is not true to-day."

Has Mr. Llano read this sentence hastily and misunderstood its author as asserting that boys of twelve or fourteen, if fitted out with the elementary knowledge in question, are competent to design and undertake engineering work, or that they could be quickly rendered competent to do so?

I call attention to the words italicized above, and submit that, properly read, the statement is true, even to-day; for, as Edgar A. Poe has well said:

"Abstruseness is a quality appertaining to no subject per se. All are alike, in facility of comprehension, to him who approaches them by properly graduated steps. It is merely because a stepping-stone, here and there, is heedlessly left unsupplied in our road to Differential Calculus, that this latter is not altogether as simple a thing as a sonnet by Mr. Solomon Seesaw."

Yours truly,

John C. Trautwine, Jr.

"Practical" Engineering.

Sir: The article by Mr. Llano in the Aug. 19 number of Engineering News contains more truths, more forcibly and fairly stated, than are contained in any other article on the same subject that it has been the writer's pleasure to read. The number of so-called "practical" men who are attempting to do correctly the work rightly belonging to men who have given the subject intelligent study is legion; literally "the woods are full of them."

Certainly there is a field for labor and a task worthy of the concentrated efforts of those who believe in good work who will undertake to lead the engineering students out of the labyrinth of false theories regarding engineering practices into which they have been led by the "practical" man, and if, while doing this, the public —the people who pay—can be delivered from this bugaboo of "practicality" those responsible for the deliverance will have done a work second to none in importance to the industrial life of this country. The paper of Mr. Llano should be copied far and wide not alone among the technical journals but in magazines which circulate among the class of people it is desired to reach.

The two following instances of poor work have recently come under the observation of the writer and while they do not belong strictly to works classed as engineering yet they are closely allied thereto and were done by men who are typical of the class that Mr. Llano complains of.

A business building costing about $25,000 has concrete foundations. The contractor, who designed the building, figured up the steel for the reinforcement of the foundations by the following interesting formula: "*For each foot in width of footing use one square inch of steel.*" This "formula" he "had found to be about right." The mystery as to how he had contrived to "find" this was not lessened when he was seen to place the bars in straight lengths on the loose dirt of the bottom of the foundation ditch and pour the concrete over them.

This man has handled work for large construction companies in different parts of the country and is perhaps above the average contractor in theoretical knowledge.

The other instance occurred in the construction of the same kind of a building as that mentioned above, having a steel frame and brick walls. The building, including the steel work, was designed and built by a "practical" man. The steel frame was so "designed" that when erected and all connections made (those that could be made) it would not stand alone and guys were used to support it until the brick could be built one story high. When it is stated that the walls were laid of re-used, poorly cleaned brick and lean mortar the stability of the structure can be well imagined. The builder confided to the writer that it was the first steel frame he had ever "designed." Let us hope that it is also his last.

In each of the instances cited the owners paid for first-class work and there was apparently no dishonesty practiced by the builders. The cost of the work to the builders was practically as much as though it had been done correctly. The results, however, are no different than if the contractors had been dishonest.

There is a great boast among the so-called "practical" men of their use of "common sense," but it is the experience of the writer, who has had ample opportunity to know the "practical" man and to be familiar with his work, that the chief characteristic of the men of this class is a lack of adaptability—the absence of plain common sense.

W. N. Collier.

Santa Rosa, Cal., Aug. 25, 1909.

The Technical Examination of Boiler Waters.

Sir: Mr. Alex. B. Moncrieff, Commissioner for Railways of South Australia (formerly Engineer-in-Chief) has kindly drawn my attention to the publication of his communication in your valuable issue of May 20, 1909, and to the comments made thereon by Mr. Stabler.

It is evident that there is very little real difference of opinion between Mr. Stabler and myself as regards the main issues, but the stumbling block to Mr. Stabler is the fact that our present knowledge with respect to the constitution of the salts in water is so imperfect that we are still obliged to make some assumptions and try to guess at the real truth.

As pointed out in our paper, printed in your issue of May 20th, the practical work of examining water can be conducted without making any assumptions whatever, and as Mr. Stabler himself proves, the data that we give correspond with his and enable the treatment of the water to be made equally as well. He, however, implies that whatever use our mode of analysis may be to a chemist, it is not so satisfactory to an engineer. As a Bachelor of Civil Engineering of some years' standing and one who has had practical training as an engineer in addition to that of a chemist, I claim to be in a position to regard this question from the view points of both these professions, and I still maintain (as originally expressed in a departmental report) that in many ways the method of analysis devised by me and worked out in practice with the assistance of Mr. Rowe is simpler than that proposed by Mr. Stabler and gives much, if not more, information. Mr. Stabler has misquoted this opinion and made me say that my "statement of combinations gives as much information," etc., which is quite a different thing.

Nevertheless, Mr. Stabler's objections to our "statement of combinations" are very considerably overdrawn. He states that a "thousand chemists might send in fifty different forms of statement." I imagine it would take Mr. Stabler a great many years to evolve fifty different forms of statement from the data supplied. He has only mentioned three forms, and of these, as I will show, only one is possible, and that is the one given in our paper.

He says that "the simple analysis cited by Mr. Hargreaves might appear in the following forms, each of which would have its staunch adherents in this country":

	(Grains per Imperial gallon.) (1)	(2)	(3)
$CaCO_3$	12.0	0.0	0.0
$MgCO_3$	5.9	16.1	0.0
Na_2CO_3	0.0	0.0	20.2
$CaCl_2$	0.0	0.0	0.0
$CaSO_4$	0.0	16.3	16.3
$MgCl_2$	2.7	2.7	21.0
$MgSO_4$	14.6	0.1	0.1
Na_2SO_4	0.0	0.0	0.0
NaCl	69.6	69.6	47.2

And he says that "when looking at these parallel columns we can readily see why Dr. F. W. Clarke, one of the foremost chemists of America, refers to a statement of such combinations as 'a meaningless chaos of assumptions'."

I quite agree with the quotation from Dr. Clarke as applied to the three columns, but I cannot agree with the contention of Mr. Stabler on this point for he has merely set up an imaginary defect in order to criticize it. If our method is carefully studied it will be seen that it proves that there is no sodium carbonate in the water, and that therefore column (3) is impossible. Mr. Stabler has admitted this.

Column (2) is equally impossible because in actually making the analysis it can be observed that in the formation of the total solids the calcium carbonate is first precipitated with a small quantity of magnesium carbonate. It is therefore impossible to believe that there can be no calcium carbonate in the water, as would be the case if column (2) were true. It is also an easy matter to show by experiment that if the water contains $CaSO_4$ and $MgCO_3$ as indicated in column (2), immediately on boiling the water in the preparation of the total solids or in the boiler of the locomotive, calcium carbonate is precipitated, being the least soluble salt. Consequently it is impossible that column (2) can represent the total solids.

Column (1) satisfies the analytical data, and is otherwise reasonable. It is therefore highly probable that the assumption upon which it is based is near the truth.

Mr. Stabler also joins issue with me when I say that "the way in which chlorine is combined makes a con-

siderable difference" but agrees that a solution of magnesium chloride is certainly more harmful to a boiler than a solution of sodium chloride. He assumes that the difference is due to the "difference between the properties of magnesium and sodium ions." The weight of evidence is against this assumption when made in the direct form stated. If it were correct, then on Mr. Stabler's theory magnesium carbonate would be equally as harmful to a boiler as magnesium chloride. It is far more probable that it is only correct in an indirect way owing to the formation of insoluble magnesium hydroxide. The magnesium chloride is probably split up at the temperature and pressure of the boiler, and the magnesium ions combining with dissociated water precipitate magnesium hydroxide and interfere with the concentration of the chlorine ions so that these acid ions (potential hydro-chloric acid) in turn combine with the iron of the boiler plate giving rise to corrosion. As sodium chloride does not form an insoluble compound its reaction in a water does not appear to lead to corrosion. It is therefore of the greatest importance to know if possible in what form the salts exist in a boiler water.

The remainder of Mr. Stabler's criticism on this point is beside the mark for it is based on column (3) referred to above, and as this column is shown to be impossible, the argument fails accordingly.

There are two small mistakes in our paper as originally printed and which were overlooked in the copy forwarded to the Engineering News. I take this opportunity of correcting them:

(1) In determining chlorine, the indicator is stated to be $K_2Cr_2O_7$, whereas we always use K_2CrO_4.

(2) The symbol "do." under "in terms of $CaCO_3$ per gallon" (see foot of page 557) is printed by mistake opposite the line Total Solids = 113.2.

Yours faithfully,
W. A. Hargreaves,
Government Analyst.

Marine Board Laboratory, Adelaide, South Australia, July 19, 1909.

A Radical Departure in Feed Water Heaters for Use in Connection with Exhaust Steam Heating or Drying Systems.

By J. W. GAMBLE.*

At the first introduction of the open type of feed water heater, trouble was encountered from the oil carried in the exhaust steam. The discovery, however, that the oil could be separated mechanically from the steam in a most efficient manner, the perfection of the apparatus for removing the oil from the exhaust steam, and the establishing of the proper proportions of this device, have led to the extensive adoption of the open feed water heater in recent practice.

There are inherent advantages in this method of heating feed water by direct contact which cannot be approached by the closed feed water heater, no matter how excellent the design of the latter. The accumulation of oil on one side of the metallic envelope and the accumulation of scale on the other and the inability to liberate the air and gases from the water have proved an insuperable barrier to the obtaining of the highest efficiency with the closed type of apparatus. Further, in connection with exhaust steam heating systems, the open heater assumes the duties of return tank or hot well, as well as expansion tank, and is usually arranged to regulate automatically the introduction of raw make-up water—functions impossible to the closed type of heater.

In the introduction of the modern high-class open feed-water heaters, numerous cases arose which called for the use of a separator to purify the surplus exhaust steam passing to the heating system, equivalent, let us say, to 500 lbs. per minute, while the exhaust steam for heating the water required in the heater possibly called for 75 lbs. of exhaust steam per minute, or less.

It was usual to meet these conditions by furnishing a heater of sufficient size to heat the water for 1,000 HP. of boilers, this heater being equipped with a separator sufficiently large for purifying the exhaust steam required for heating this water. The surplus exhaust steam, that is, the steam not required in the heater, would be passed through an independent oil separator on its way to the heating system in order that the latter might not be fouled by oil and so that the condensed returns would be suitable for boiler feed purposes.

In many cases, however, engineers, owners and architects, called for provision for by-passing the heater, should it be found necessary at any time to cut the heater out for examination, cleaning or repairs. This led to the adoption of an expensive arrangement of by-passing, which, besides its first cost, frequently occupied room which could ill be spared; increased the number of joints in the pipe lines; increased the pipe covering required; and in some cases endangered the heater because of an improper operation of the cut-out valves. In some cases it necessitated a larger engine room or boiler room than would otherwise have been required.

These conditions were acquiesced in, however, as representing the best engineering practice, nothing having been offered which would enable the engineers to avoid the troublesome features enumerated above. Where the piping conditions would have been extremely expensive and troublesome when using the independent oil separator arrangement, a larger heater than would otherwise be required was sometimes installed for the sake of the larger separator which the larger heater carried, the engineers accepting this alternative as being the better combination.

The idea, however, continued to persist that some way should be devised to effect a simplification of the problem. In the course of the development of the idea it was suggested that by attaching what was the equivalent of the independent oil separator to the heater body, and taking the surplus steam away from the oil separator at a point exterior to the heater, providing suitable cut-out valves in the exhaust inlet to the heater and in the waste and overflow connection, a device would be produced which would represent a much desired consummation and one which would be eagerly accepted and its value appreciated by the consulting and operating engineers and by architects.

The first heater of this design was placed in service a little over a year ago by the Harrison Safety Boiler Works, of Philadelphia, and its valuable features became at once quite apparent.

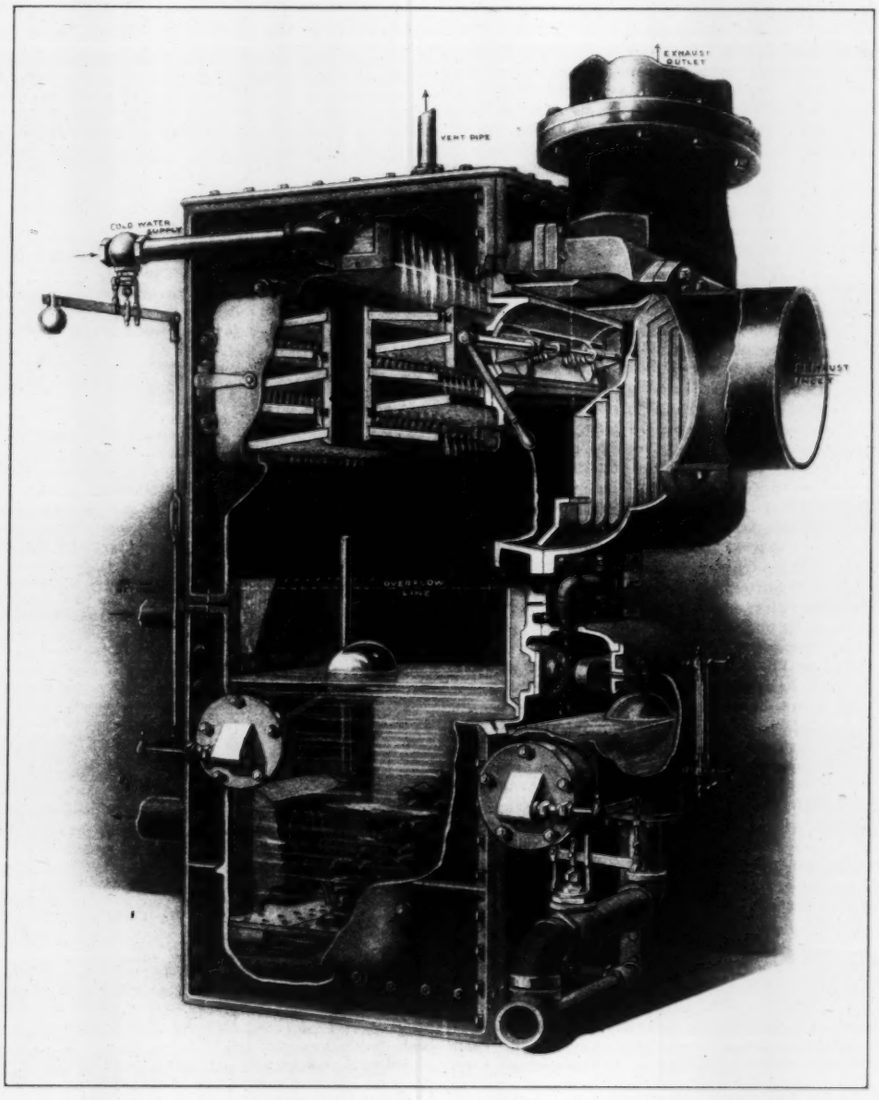


FIG. 1. AN OPEN FEED-WATER HEATER WITH STEAM SEPARATOR ATTACHED.

The sectional cut presented herewith shows the construction of the heater proper and requires no explanation to those skilled in the art. It should be pointed out, however, that when the cut-out valve controlling the admission of steam to the heater and the overflow valve controlling the overflow from the heater are closed, the separator continues to purify the exhaust steam which may be passing to the heating or drying system, and that the trap which drains the separator is also in service at all times. Therefore this combination enables the user to dispense with one independent oil separator and trap for draining the same, and one, two or three cut-out valves for by-passing the heater, according to the arrangement adopted for by-passing.

The line drawing shows the manner in which the heater is connected up to the engine and the heating system.

*Engineer, the Harrison Safety Boiler Works, Philadelphia, Pa.

In the work of invention, it is a paradox that the obvious is often not apparent. The failure of feed water heater manufacturers for a long period of years to size up the possibilities offered by a proper combination of the apparatus required in a modern feed water heater, and their failure to produce apparatus of adequate design for securing the valuable features, illustrates the pertinence of this observation.

Proceedings of the Engineering Section of the British Association for the Advancement of Science, Winnipeg Meeting.

By GEORGE W. BISSELL,* M. Am. Soc. M. E.

The British Association for the Advancement of Science held its 79th annual meeting in Winnipeg, Man., during the week Aug. 25 to Sept. 1, its fourth meeting outside of the British Isles. The previous foreign meetings had been at Montreal, Toronto and Cape Town; the last meeting on this continent was in Toronto in 1897.

The Engineering Section of the Association held five sessions. Their proceedings are briefly reviewed in the following. The section was headed by Sir William H. White, whose presidential address fittingly inaugurated the meeting. He reviewed at length the progress and achievements of Canada in railways, waterway improvement, navigation and water-power development, setting forth the rapid development which Canada has accomplished in all these lines of activity.

"Hydroplanes or Skimmers," by Sir John Thornycroft, was read by the secretary, the author being absent. The paper† reported the results of experimental work on models and small vessels. These investigations have immediate bearing on the design of motor-boats, in which at high speed there is frequently a tendency to dancing or plunging, which may be very dangerous. Mr. Thornycroft obtained very satisfactory results, so far as steadiness is concerned, with a 22-ft. boat which was fitted with

a small plane attached to a downward extension of the stem or forward deadwood, parallel to the bottom surface of the boat at its center or greatest section, but about 1-in. below this surface; and this although offering little resistance at starting, as the water can flow over it, affords the necessary stability to keep the boat at the right angle for skimming.

Mr. C. B. Smith presented a paper "The City Hydro-electric Plant." This plant, now under construction, is located on the Winnipeg River at a distance of 77 miles from Winnipeg. It is expected to have an ultimate capacity of 60,000 HP., of which 20,600 HP. will be installed at present. The available head is 45 ft. The generator units are of 3,000-KVA. capacity each, and generate 60-cycle three-phase current at 6,600 volts, which will be stepped up to 66,000 volts for transmission. The generators will be direct-connected to 5,200-HP. double horizontal submerged turbines operating at 164 r. p. m. The transmission line will be of aluminum, supported on steel towers. The estimated cost of the undertaking is $3,000,000. Mr. Smith is construction engineer and Col. H. N. Ruttan is Chief of the Board of Consulting Engineers.

Lt. Col. W. P. Anderson, Chief Engineer of the Department of Marine and Fisheries, read a paper, "Improvements in the Navigation of the St. Lawrence," which gave a very clear account of the improvements in the channel and the means for maintaining the depths; of lighthouses and other aids in navigation, including submarine bells (which he considers very desirable and efficient aids of communication); of surveys and hydrography; of ice conditions and winter navigation; and of the ports from Montreal to the sea. A 27-ft. channel is available from the open sea to Montreal.

Maj. G. W. Stephens, President of the Harbor Commissioners of Montreal, presented a paper entitled "The St. Lawrence River, the Great Imperial Highway of Canadian Transportation," in which he stated and emphasized the importance of Montreal as a shipping point for Canadian products. In connection with these papers President White presented a summary of the government report on the Georgian Bay Ship Canal, designed to connect Georgian Bay with the Ottawa River by a 20-ft. channel route about 400 miles long.

The most important paper of the entire program was by Col. G. W. Goethals, Chief Engineer and Chairman of the Isthmian Canal Commission, on "Engineering and Construction Features of the Panama Canal." The paper was presented by the author's son, Lieut. G. R. Goethals, U. S. A., who has returned from 14 months' service in the Canal Zone. The paper dealt at length with the Gatun dam, the locks and the Culebra cut, and gave statistics of performance and details of difficulties encountered. Lantern slides were freely used in illustration. As a special compliment to the author, as well as recognition of the value of the paper, it will be published in full in the Proceedings of the Association (each section is granted space for one paper).

"The National Transcontinental Railway," by Mr. Duncan McPherson, was read by the Secretary. The railway will eventually be part of the Grand Trunk system. The eastern division, from Moncton, N. S., to Winnipeg, Man., is being constructed by the government and will be leased to the Grand Trunk Pacific Ry. The latter corporation is constructing the line from Winnipeg to Prince Rupert, B. C. On the eastern division the maximum curve is 6°, and the maximum grade 0.4% eastbound and 0.6% westbound, compensated for curvature. The easy gradients, combined with a reduction of 420 miles in the distance from Winnipeg to Quebec as compared with the shortest existing rail route, are expected to give the new line considerable advantage and to justify its construction.

Mr. J. E. Schwitzer, Asst. Chief Engineer of the Canadian Pacific Railway, Western Lines, in a paper entitled "Two Great Engineering Works on the Canadian Pacific Railway," described the Lethbridge Viaduct and the Kicking Horse Pass improvement, both just completed.

The Lethbridge viaduct is 5,327 ft. in length, and some of the steel towers on which the spans are supported are 285 ft. high beneath the gird-

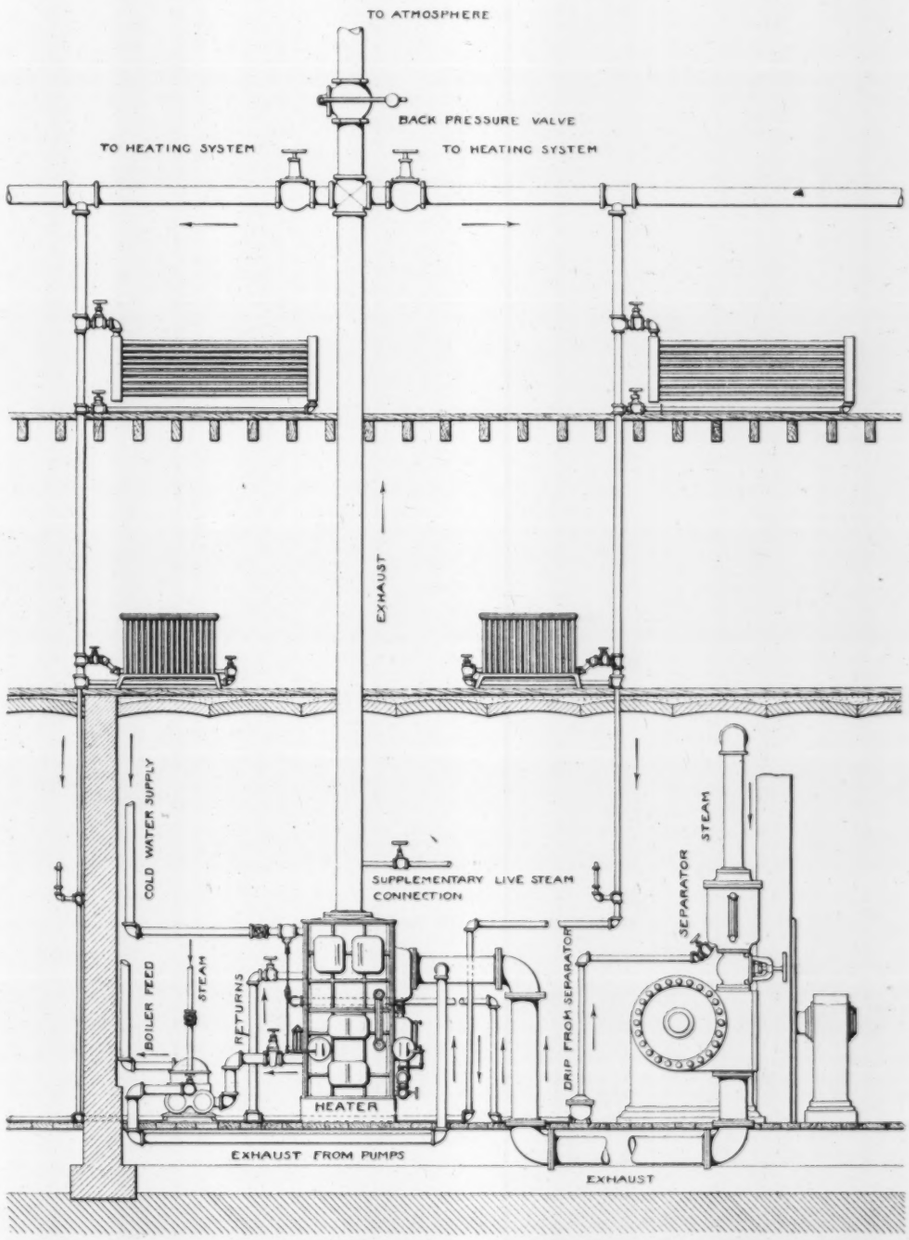


FIG. 2. SECTION OF BUILDING SHOWING CONNECTION OF FEED WATER HEATER TO PIPING.

*Dean of Engineering and Professor of Mechanical Engineering, Michigan State Agricultural College, East Lansing, Mich.

†A paper of similar title but more general in its nature, by the same author, was abstracted in our issue of July 8, 1909, p. 53.—Ed.

ers. It contains 12,200 tons of steel. Owing to the rugged character of the site and its remoteness from the base of supplies the difficulties of construction were great.

The original line of the Canadian Pacific between Hector and Field, B. C., contained a section of 4.1 miles on 4.5% grade. Four engines were required to haul a 700-ton train over this grade. The recent improvement gives 2.2% maximum grade through the Kicking Horse canyon. It involved building two loop tunnels, 2,912 and 3,184 ft. long respectively, and one short tangent tunnel.

Col. H. N. Ruttan described the high-pressure water service, Winnipeg, of which undertaking he was the engineer. The plant consists of four 540-HP. and two 250-HP. Crossley two-cylinder tandem gas-engines each geared to triplex pumps. The engines operate on producer gas. City gas can also be used in emergency. The suction and discharge mains are 20 ins. in diameter. The capacity of the entire plant is 9,000 gals. per minute against a pressure of 300 lbs. per sq. in. Gas from the producers is stored in a holder of 250,000 cu. ft. capacity. The amount invested is $1,600,000, raised by bonds whose interest charges are used as the sinking fund and cost of operation are met by tax on the property protected; but the tax is less than the annual saving in insurance (estimated at $200,000). This undertaking, as well as the municipal hydro-electric plant described in Mr. Smith's paper and other large public undertakings, is evidence of the city's enterprise and faith in its future, which is rather remarkable when the present population, 130,000, is considered.

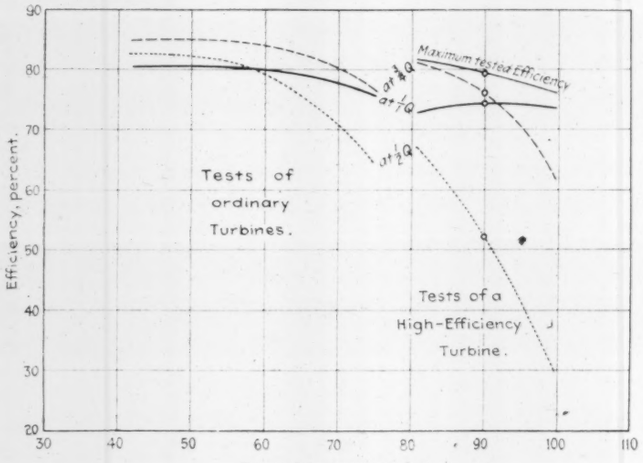


FIG. 1. VARIATION OF EFFICIENCY WITH TYPE CHARACTERISTIC OF TURBINES.

(The curves at the left are taken from tests of standard German turbines. The curves at the right represent the test charted on base of discharge in Fig. 3.)

"The Development of the Grain Industry of Western Canada and its Future Possibilities," a paper by Mr. G. Harcourt, and "Grain Handling," by Mr. W. B. Lannigan, opened the eyes of the audience to the vast magnitude and growth of the grain business in Manitoba and the other western provinces. The industry is a continual surprise even to citizens of the United States, and the extent of untouched but potent areas of wheat land in the Dominion is overwhelming. In 1908, the three provinces of Manitoba, Saskatchewan and Alberta produced 103,900,000 bushels of wheat, 108,980,000 bushels of oats and 24,000,000 bushels of barley, the respective areas under cultivation being 6,760,000, 3,420,000 and 965,000 acres. In the same provinces there were, on May 1, 1909, 1,490 elevators with aggregate capacity of 44,000,000 bushels.

Prof. E. G. Coker read in abstract a very complete paper reporting the extensive and thorough investigations of Mr. C. E. Larrard on "The Behavior of Ductile Material Under Torsional Strain." The tests were made on large sections, up to 3 ins. in diameter, and the author lays stress on the importance of testing under conditions approximating as nearly as possible those of service, as to both magnitude and character of the stress.

Prof. J. B. Porter, of McGill University, made a preliminary, informal report of the fuel tests being carried out by him for the Dominion Government. The work is intended to furnish users of Canadian coal information like that obtained by the U. S. Geological Survey for the coal consumers in this country. Particular stress is being placed in boiler tests in boilers of medium size, 50 to 60 HP., so that the results may be useful to the average user rather than to the larger plants which in general have ample facilities for doing their own testing.

Other papers presented before the meeting were:

"The Distribution of Dielectric Stress in Three-Phase Cables," by W. M. Thornton and O. J. Williams, of Newcastle-on-Tyne.

"The Losses from Overhead Direct-Current High-tension Lines due to Brush Discharge," by E. A. Watson, of Birmingham.

"The Calculation of Charging-currents in Three-core Cables and Three-phase Overhead Transmission Lines," by Prof. E. W. Marchant, of Liverpool College.

"International Electrical Standardization," by O. Higman.

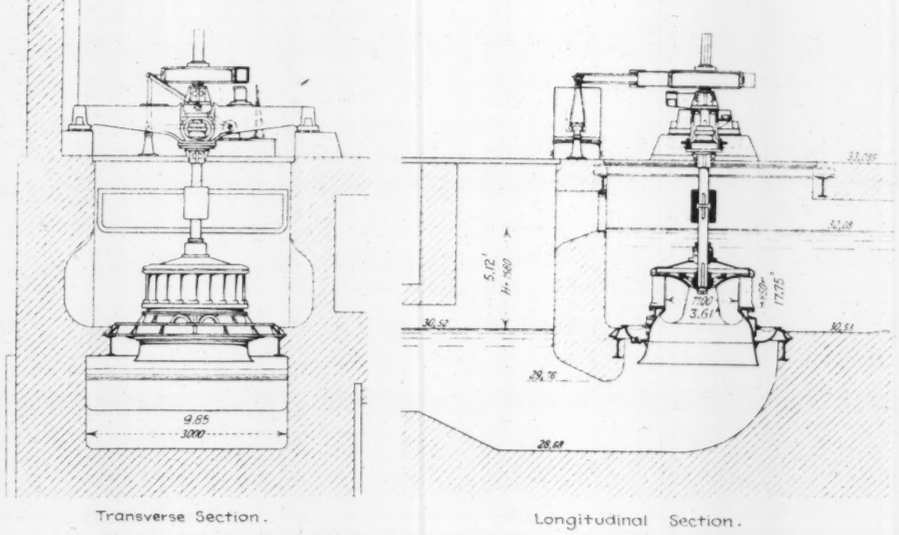
The latter was a review of the standardization so far accomplished, while the other three papers were reports of elaborate and thorough experimental work in their respective subjects.

The section's committee on Gaseous Explosions, which is investigating the behavior of the explosive charge in internal combustion engines, reported progress. Mr. Dugald Clerk is at the head of this committee; Prof. E. G. Coker read the report.

The sessions above reviewed were held on the mornings of Thursday, Friday, Monday, Tuesday and Wednesday (Sept. 1), leaving the afternoons of these days and all of Saturday for other features. This time was occupied chiefly by excursions to neighboring points of engineering interest. Among them were the yards and shops of the Canadian Pacific and Canadian Northern railways, the high-pressure fire-service pumping plant, the municipal hydro-electric plant, St. Andrews lock on the Red River, etc.

The evenings were devoted in part to popular lectures, but more largely to various social functions, in which the citizens of Winnipeg showed their hospitality in a very generous and delightful manner.

The meeting is followed and supplemented by an excursion to the Pacific Coast on special train, visiting important points of western Canada, as Regina, Moose Jaw, Calgary, Edmonton, Banff, Laggan, Vancouver and Victoria.



Transverse Section. Longitudinal Section.

FIG. 2. SPECIAL HIGH-EFFICIENCY TURBINE.
(Designed by Wagenbach. Built by J. M. Voith.)

Tests of a High-Efficiency High-Speed Hydraulic Turbine Runner.

By ERNST REICHEL.*

In the issue of Jan. 28, 1909, of this journal, Mr. S. J. Zowski published an investigation in which the characteristic features of a water-turbine as to discharge and speed are discussed in the European style. For a turbine let H signify the head, H-P the horsepower and n the number of revolutions per minute. Then the formula

$$K_t = \frac{n}{H}\sqrt{\frac{H\text{-}P}{\sqrt{H}}}$$

expresses the comparative speed of a turbine. In Europe this expression—the "type characteristic" of Zowski—is called the "specific speed," according to the proposal of Prof. Camerer, of Munich. Turbines are therefore distinguished as

		Foot system.	(Metric system.)
Slow-speed runners,	with K_t =	13.5 to 18	(60 to 80)
Medium-speed runners,	with K_t =	34 to 36	(150 to 160)
High-speed runners,	with K_t =	55 to 80	(250 to 350)

Standard turbines with other than above specific speeds are seldom built.

American engineers have for a long time past stopped at the value K_t = 80 (350) as the upper limit of specific speed, while in Europe no one wished to build any higher than K_t = 72 (320), because it was found that the increase of specific speed causes a decrease of the actual efficiency of the turbine at the normal number of revolutions. This relation can be very clearly shown by a diagram whose abscissas are the specific speed or type characteristic K_t while the ordinates are the efficiencies. The left-hand part of the chart Fig. 1 show such curves, the figures used for them being taken from tests of turbines manufactured by good European factories.

With regard to this relation the question arises whether it should not be possible to raise the efficiency of high-speed runners still further by better construction of the buckets. On the other hand the question arises to what extent the specific speed of turbines could be increased without decreasing the efficiency too much. In order to study these relations a turbine has re-

*Professor at the Technical College of Charlottenburg, Germany.

cently been tested with the utmost care in the laboratory for water-motors in the Technical College of Charlottenburg which is under the direction of the author. The turbine, shown in outline by Fig. 2, had been designed for the following conditions:

Head, 5.12 ft. (1.56 m.)
Discharge, 88.4 cu. ft. per sec. (2.5 cu. m. per sec.)
Runner diameter, 43.5 ins. (1,100 mm.)
Height of gate, 17.5 ins. (450 mm.)
Speed, 112.5 r. p. m.

In the tests the discharge of water was measured most exactly by a new method, "with screen."*

The results of the tests are shown in Fig. 3. The numbers of revolutions n_1 are for a head

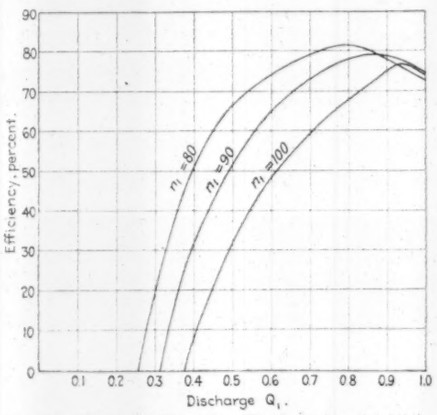


Fig. 3. Result of Efficiency Tests of Special High-Efficiency Turbine.

$H = 1$ m. The maximum discharge is represented by $Q_1 = 1.0$ cu. m. per sec. It will be seen that the turbine was tested under three different speeds, with the result given in the table below:

R. p. m.	100	112.5	125
Corresponding specific speed K_t	79.4 (354)	90 (400)	100.2 (445)
Maximum efficiency.	81.6%	79.2%	76.8%

It will be seen from the tests that the curves of efficiency for constant speed and variable discharge become more and more steeply sloped with increasing specific speed.

These figures go to show that the maximum speed of turbines has not yet been attained and that high-speed runners may be designed surpassing those known up to date and giving in spite of that a very satisfactory efficiency. These turbines are especially adapted to low heads.

The high-speed runner tested at Charlottenburg was designed by Herr Wagenbach, Engineer at the Technical College, and was built at the J. M. Voith Works, Heidenheim, Germany.

Rate Decisions of the Wisconsin Public Utilities Commission.

By CLINTON ROGERS WOODRUFF.†

The public utilities commission of Wisconsin has handed down several decisions likely to prove of far-reaching importance. The theory underlying the Wisconsin public utilities law is that rates shall be reasonable, and, at the same time, yield a fair or reasonable return on the investment, the commission being given the power to fix the rates to secure these ends. It has been called upon to determine what the investment actually is, and on one phase of this it has rendered a decision to the effect that, generally speaking, it is undoubtedly true "that the greatest amount of justice is done when abnormal conditions are eliminated," and that "a plant should be given no greater consideration when unreasonably high than when it was had at a bargain"; certainly a common-sense view to take. The commission's conclusions on these matters hold in effect that the value to be allowed a public-utility corporation must be closely associated with cost, with investment made with reasonable wisdom, and may not include any fictitious figures set up by the companies as value and supported on the ground of earnings, or on the ground of manifestly unwise expenditures, or yet on the ground of fictitious costs or liabilities involved in "expert" financiering.

Some of the corporations claimed that the franchises which they had secured from the cities should be taken into consideration in fixing the value and the rates, the same as machinery, buildings and other physical property.

In the case of Payne and others of Marinette against the Wisconsin Telephone Co., involving the charge that telephone rates at Marinette were excessive and that they had been unlawfully raised, the commission ruled that no franchise value can be allowed in Wisconsin as attaching to telephone properties for the purpose of fixing rates. While good-will value might be allowed, owing to the contemplation of telephone utilities as competitive enterprises, rather than as monopolistic enterprises, as shown by the Wisconsin public utilities law, no good-will value will be allowed for rate-making purposes unless the competition is actually present, not merely possible through the chance that a competitive telephone exchange may be established.

The commission recognizes a value corresponding to what the corporations generally claim under the head of "going concern" value or cost of securing an established business, but none was allowed in the Marinette case, the commission on this point saying:

> The value of a going concern is generally greater than the sum of the values of the separate physical parts of the plant. The seller of such a plant is in a position to exact more and the purchaser would generally be willing to pay more than for a plant which has no established business. In expropriation proceedings, likewise, the owner or owners of a plant which is a "going concern" would doubtless be awarded a larger amount of damages than the owner or owners of a plant which has not yet been placed upon a going basis. But this "more" in the value of a plant in the case of purchase and sale or expropriation is not a matter in which the public is interested in proceedings of this kind. This "more" is not property used and useful for the convenience of the public within the means of the statute. On the other hand, if property is devoted to the public use and reasonable care has been exercised in all the phases of its management, but the owners have not yet received a fair return during the earlier years of the operation of the plant in which the property is used for the convenience of the public, the deficits thus incurred must be made up out of later earnings in so far as this is commercially possible and expedient. In other words, every effort honestly put forth, every dollar properly expended and every obligation legitimately incurred in the establishment of an efficient public utility business must be taken into consideration in the making of rates for such business. Collectively, the elements just referred to may be designated by the term going value and in this sense there can be no question regarding the property and justice of admitting going value as a consideration in the determination of rates. Whether this going value should be made a part of the permanent capitalization of the plant or provided for by means of a sinking fund or other fund is a matter to be decided on the facts of each particular case. If it is made a part of the capital investment, it becomes a permanent charge against consumers for all time to come. If it is provided for by means of a sinking fund, all persons who are patrons of the plant during the period in which the sinking fund is accumulated will alone contribute.

Certainly, as the Milwaukee "Journal" points out, these decisions "wipe out millions of values," because the commission has placed the stamp of disapproval upon the claim of the corporations for large "intangible franchise value," simply by accumulating expert testimony to the effect that a franchise for which nothing was paid is worth a large sum as a basis for fixing the rates which the public may be charged for service.

It will be interesting to see how this far-reaching series of opinions will be accepted, although a new attitude is being assumed by investors towards commissions, which was reflected in the address by President Frank A. Vanderlip of the City National Bank of New York before the American Electric Light Association meeting at Atlantic City, in which he pointed out that we are, on the whole, a very sensible people. We believe in initiative, and do not care to have great business enterprises dominated by red tape. The public wants fair play, and is in a position to demand and get it. Intelligent management of properties will recognize that those that give freely to their customers good service at reasonable cost will, I believe, have little cause to complain of unfair treatment at the hands of legislators or commissions. The commission that demands only fair and reasonable treatment of the consumer, and in return secures the corporation from piratical attack of competitors organized only to be bought out will, in the end, prove a bulwark to the security holder.

*See Zeitschrift des Vereins deutscher Ingenieure, Nov. 14, 1908, page 1839.

†Secretary National Municipal League, North American Building, Philadelphia, Pa.

A Recent Trouble with Large Freight Cars.

The maximum size of freight cars was a frequent subject for discussion at meetings of railway associations a few years ago, and it still crops up occasionally, mainly in regard to the difficulties of operating very large cars within the clearance limits of some railways. These very large cars are built usually by western lines, and trouble is apt to arise if they are forwarded over eastern lines, many of which have comparatively small clearance limits for some of their older tunnels and other structures. At the recent annual meeting of the Association of Transportation and Car Accounting Officers, this question was brought up by Mr. T. S. Bell, of the Pennsylvania R. R., who stated that his road has had trouble with some very high box cars belonging to western lines. The maximum height of Pennsylvania R. R. box cars is 13 ft. 4 ins. from the rail to the running board, but the height of these foreign cars was 15 ft. 7 ins.; this was too high for some of the close points, such as signal bridges. He remarked that the same trouble would be experienced on many other eastern railways, and the association voted that the matter should be investigated by the executive committee.

A few years ago the Master Car Builders' Association recommended maximum dimensions for box cars, which included a length of 36 ft. (inside), a width of 9 ft. 7 ins. at the eaves, and a height not exceeding 12 ft. 6¾ ins. from the rail to the top of the eaves. This last figure would give a total of a little over 13 ft. to the top of the running board, but in some large cars the height is 14 ft. or more. It is evident that 15 ft. 7 ins. is an extreme height, and it appears probable that the cars in question are special cars built for carrying a full-capacity load of bulky but light material, such as furniture and carriages. Such cars are operated by a number of western railways, but they are relatively few in number, and their use is largely confined to the home lines.

In the design of tunnels, overhead bridges, signal bridges, etc., engineers very often may have to consider the relations of standard clearance limits to the extreme dimensions to be provided for in these structures. It is not desirable to block traffic by refusing to accept large cars in interchange service, but on the other hand it may add excessively to the cost of the work if it is made to accommodate the cars of the maximum size (which may never be operated over the particular line under consideration).

RUSSIAN IRON AND STEEL production for the first eleven months of 1908, in comparison with the same period of 1907, was recently given by the British Consul at Mariupol as follows:

	1907. Tons.	1908. Tons.
Pig iron	1,628,096	1,722,587
Ingots	1,197,410	1,220,520
Billets and blooms	455,666	509,234
Iron and steel (other kinds)	896,153	942,175

The Vice-Consul of Nicolaieff reported that the quantity of iron ore exported in 1908 showed a considerable decrease on that of the two preceding years, amounting to only 215,000 tons, as against 511,763 tons in 1907 and 267,693 tons in 1906. The export of rails, on the contrary, showed an increase, the total quantity shipped from the port being 24,260 tons, as against 16,245 tons in 1907. A noticeable feature of the trade in 1908 was that a wider market was found, rails being exported to India, Japan, Roumania, and even to the United Kingdom, whereas in 1907 Italy was practically the only buyer. This extension seems likely to continue, orders having been placed recently for Dublin and Cape Colony. The following table gives the various countries of destination, with the respective shipments in 1908: Japan, 7,050 tons; India, 6,047; Roumania, 5,241; Italy, 2,928; United Kingdom, 2,001; Belgium, 501; Netherlands, 492; total, 24,260.

Pressure of Concrete on Forms.*

By FRANCIS R. SHUNK.†

During the season of 1908 experiments were made at Lock No. 1, on the Mississippi River, between St. Paul and Minneapolis, with a view of determining the pressure of concrete on forms at various temperatures and rates of filling.

The lock walls are made up of monolithic blocks 25.75 ft. high. Each block was built in a single working day, so that there are no horizontal joints. The ingredients of the concrete were mixed about in the proportions of 1:3:5½, and the mixture was very wet, so that the

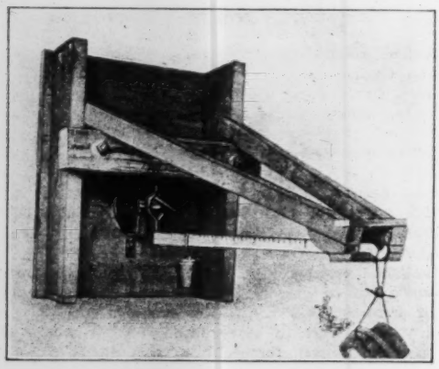


Fig. 1. Apparatus Used to Determine Pressures in Concrete Forms.

forms had to be water-tight, and the workmen sank in about 18 ins. The weight was 152 lbs. per cu. ft. There was therefore considerable pressure on the forms. As it was not known beforehand just what this pressure would be, the forms were designed by good guesswork or judgment, and answered very well; but measurements were made for future use.

The necessity for definite knowledge was brought forcibly home by the fact that, when cold weather set in, some timbers broke in one of the forms which had been successfully used all summer; and, again, after alternate monoliths had been completed and we were filling in the intervals, one of these great blocks, on a fine, bracing, autumn day, began to tip over. The experiments were therefore far from superfluous.

The measuring apparatus is shown in Fig. 1. A cylinder 9.23 ins. in diameter, fitted with a piston made of two ¼-in. metal plates with ½-in. of leather between, was fastened into the side of the form about 18 ins. from the bottom. Although the piston was well greased with a heavy oil for the first experiments, grit found its way to the leather, which accounts for the large initial pressure required to move the piston on Aug. 27 and Sept. 14 and 18. After the latter date, the end of the piston was covered with several inches of soap. This made it tight and practically frictionless, as shown by the later observations. The weight arm or beam of the gage is about 5 ft. long and is graduated in feet and tenths. The constant of reduction was not known to the observer who took the readings. His records gave merely the weight leverage, from which the pressure per square foot was figured in the office.

Concrete not being a freely moving liquid even while exerting liquid pressure, it was impossible to obtain absolutely accurate pressure measurements. To meet this difficulty as far as possible, an adjustable pivot was put in between the arm and the piston. By means of this the piston could be moved independently of the weight arm. At each pressure determination, the weight leverage, having been previously increased, was decreased until the concrete pushed the piston out and raised the weight arm. After recording the weights and their lever arm, the weight leverage was increased until it was considerably greater than the concrete pressure leverage, and the arm was lowered by turning back the pivot.

When the form had been filled another half foot, the next determination was made in the same way. Before the readings it was usually necessary to move the piston in and out about a quarter of an inch until the observer felt it to be "alive" (as was said). The recorded pressures are those at which the concrete followed up, i. e., actually moved the piston out against the weight leverage. They are very much smaller than those represented by the weight leverage required to bring the arm down. It was to avoid this pressing in against the concrete that the adjustable pivot was inserted. As the setting of the concrete proceeded and the arch action of the particles increased a point was reached at which the concrete would suddenly refuse to "follow up," and while there undoubtedly was still considerable pressure on the form the gage would no longer record it, and observations ceased abruptly. This point, however, was never before the maximum pressure had occurred.

The results of the experiments were plotted for a number of tests, from which diagrams a series of general curves were deduced. Of the record curves only one (Fig. 2) is shown herewith, but all the rest are similar in design; though for various temperature and rates of filling. Each record sheet (Fig. 2) contains two curves or lines. One (at the left) shows the relation between time and the height of the surface of the liquid concrete. The abscissa is time (as indicated at top); the ordinate is height above center of piston. The other line (at the right) shows the relation between pressure and the height of the surface of liquid concrete. The abscissa (as shown at bottom) is pressure in pounds per square foot. The ordinate (as before) is height above center of piston, or head of concrete. Neither of these lines is derived from the other. Both are plotted from direct observation.

The method of deducing the final curves was as follows. The pressure on the piston at first is that due to pressure of a liquid weighing 152 lbs. per cu. ft. After a time, the pressure on the piston becomes less than the liquid pressure, because the concrete begins to act as a solid, and is then equal to the liquid pressure minus a certain resistance due to setting and arch action, which (for lack of a better name) has been called strength. The time at which this strength begins to show varies with temperature, pressure, condition of rest or agitation, and, perhaps, other things. From the data available a formula for time of initial strength has been deduced, which is $T = C + 150/R$, in which T is time in minutes after filling has begun, when concrete begins to show strength, R is rate of filling in vertical feet per hour, and C is a constant depending on temperature of concrete. Values of C for the usual temperatures are as follows:

Temperature.	C.
80° F.	20
70	25
60	35
55	42
50	50
40	70

The temperature considered here, as elsewhere in this discussion, is the temperature of the mixture in the form.

It appears from the formula that the concrete shows strength more quickly as the rate of filling is greater. This is because a given layer is more quickly protected from the effects of agitation on the surface when the rate of filling is rapid than when it is slow. In the latter case a given layer may be for an hour or so kept in constant agitation by the trampling of the men, dumping of material, etc. The effect of pressure, which doubtless exists, is so disguised by the effect of agitation that it cannot be detected.

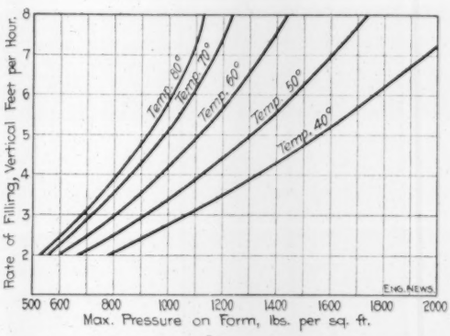


Fig. 4. Final Pressure-Rate of Filling Curve in Concrete Forms.
(For all temperatures and all rates of filling.)

[Fig. 3 is a sample of the primary deduced curves, one of which was made for each temperature. In this diagram the curves on the left are the values of strength (as above defined) plotted against time, for the various rates. These values are taken from the curves, such as Fig. 2, by plotting against time, the horizontal distances (i. e., pressures) between the curve of actual pressure and the continued line of liquid pressure. The right-hand curves on Fig. 3 are the actual liquid pressures plotted against time instead of against head as shown in Fig. 2. —Ed. Eng. News.]

The time of the maximum pressure is the ordinate of the point (marked by a short normal) on the curve S where the tangent is parallel to the line of liquid pressure for the same value of R. Its value is the horizontal distance between the curves of S and P at this time.

From diagrams like Fig. 3 one for each temperature, the curves on Fig. 4 are plotted, the values of R being entered as ordinates, the maximum pressures as abscissas. There is a separate curve for each temperature.

These curves may be used for two purposes; either to determine the strength to be given to forms, or to determine the safe rate of filling for a given form. In the first case, it is necessary to determine, from records of the locality, the lowest temperature to be expected, and, from the dimensions of the blocks to be built, the highest desired rate of filling. Suppose that the lowest tempera-

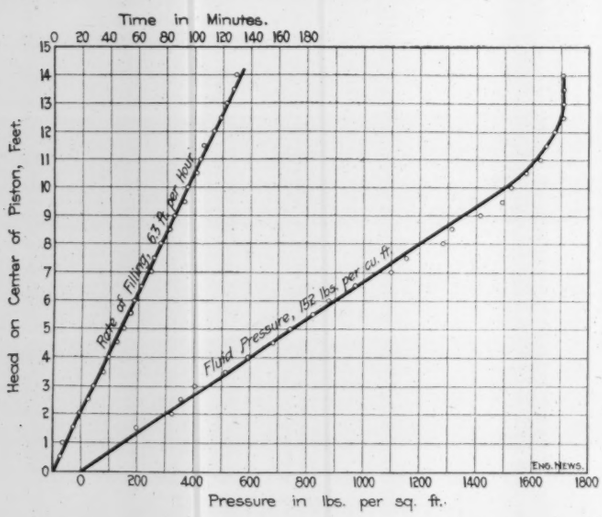

Time in Minutes.
Head on Center of Piston, Feet.
Rate of Filling, 6.3 ft. per Hour.
Fluid Pressure, 152 lbs. per cu. ft.
ENG. NEWS.
Pressure in lbs. per sq. ft.

FIG. 2. SAMPLE PRESSURE-TIME CURVE IN CONCRETE FORMS.
(For one temperature and one rate of filling.)

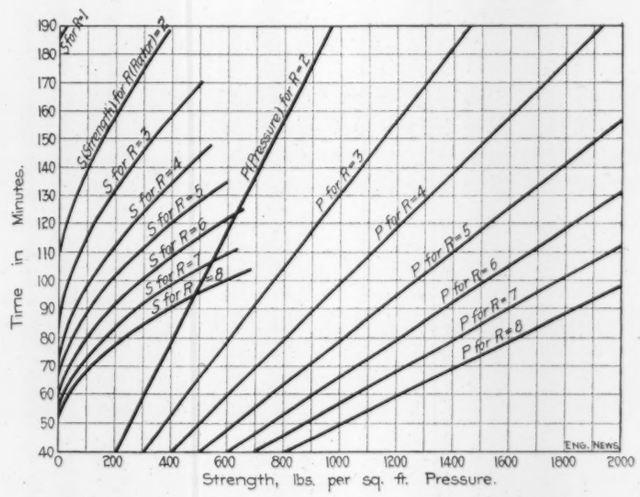


FIG. 3. PRESSURE-TIME CURVE IN CONCRETE FORMS.
(Temperature, 60° F.; various rates of filling.)

*From Professional Memoirs, Engineer Bureau, United States Army, July-September, 1909, Vol. I., No. 3.

†Major, Corps of Engineers, U. S. A.

ure is 40°, and the desired rate of filling 7 ft. per hour. Follow the horizontal line 7 until it intersects the curve for 40°, and drop a vertical to the axis of X. This shows that the forms must be designed for a pressure of about 1,900 lbs. per sq. ft. In the second case, suppose that the form is designed for a pressure of 1,000 lbs. per sq. ft., and that on a given day the temperature is 60°. Follow the vertical line 1,000 until it intersects the curve for 60°, thence a horizontal line to the axis of Y. This shows that the safe rate of filling is something over 4 ft. per hour.

I am far from claiming absolute accuracy for the results obtained, but I think they are sufficiently exact for the purpose, which is not to establish a theory of molecular physics, but only to find a practical rule for determining strength of forms or rate of filling. During the coming season I expect to use a larger piston (2 sq. ft. in area if possible) and to make a greater number of measurements.

General Conditions of Transportation by Water in the United States.*

This report deals only with domestic traffic, except so far as foreign traffic is incidentally involved in such discussion. The domestic water-borne traffic carried by vessels of the United States is far larger than the foreign trade so carried, and of much greater importance. Coastwise traffic is by law restricted to American vessels. Most of the foreign trade is carried in foreign vessels. Domestic coastwise commerce, however, uses the same ports as does the foreign commerce, and the early policy of the Government in improving these ports, with especial view to foreign trade, gave also especial advantage to coastwise trade. In physical characteristics also the coastwise business is of the same class as foreign business, while the two differ largely from inland water traffic.

Waterways and Their Improvement.

The possibilities of transportation by water in the United States may be roughly indicated by a brief survey of the extent of its waterways.

COAST LINE.—The Atlantic cost line of the United States is more than 2,0 0 miles in length, and it is extended for a little over 1,850 miles more by the Gulf of Mexico. The Pacific coast line is more than 1,800 miles long. The coast line of continental United States aggregates 5,705 miles. This is increased to a meandered length of 64,694 miles by including the numerous indentations, many of which provide important bay, sound, and inlet routes. Chief of these are the Gulf of Maine (so-called), Long Island Sound, Delaware Bay, and Chesapeake Bay, on the Atlantic coast, and Puget Sound, on the Pacific coast.

GREAT LAKES.—Next in significance are the Great Lakes, the most important group of inland waterways in the world. Their shore line in United States territory is 2,760 geographical miles. In the meandered length there is 4,329 miles. They are connected by a series of natural and artificial channels. Canals also connect the Lakes with the Atlantic Ocean and the Mississippi and Ohio rivers, but these are not of dimensions to permit of through navigation by large vessels. Another series of canals, constructed by the Canadian government along the St. Lawrence River, give 14-ft. draft to Montreal, and are used to some extent by American vessels.

On the coasts the Government has deepened harbors and connecting channels. Similar works on the Great Lakes give a depth of 20 ft. on the main channels at mean water level.

COAST AND LAKE ROUTES.—The coastwise trade routes radiate mainly from a few central ports, such as Boston, New York, Philadelphia and Baltimore, on the Atlantic coast, and San Francisco, Portland and Seattle, on the Pacific. On the Lakes, notwithstanding the large number of routes, the great movement of traffic follows a few well-defined main lines. Most important is the route from Lake Superior ports across Lake Huron to Lake Erie ports. Another main route, formerly the most important, is that from Lake Michigan ports to ports on Lake Erie.

RIVERS.—The number of navigable streams used to a considerable extent for commercial purposes in the United States is about 295, with an approximate mileage of 26,400. (See Table I.)

The streams tributary to the Atlantic do not form connected systems, but are often extensions of coastwise routes into the interior. Streams tributary to the Gulf are more often connected with each other, especially those of the Mississippi Valley system, which aggregate about 14,000 miles. On the Pacific coast there are also several connected systems, such as the rivers leading into Puget Sound, the Columbia River and its tributaries, and the rivers of central California.

Most streams included in the foregoing table are navigable only for light-draft boats. Forty streams have a total of about 2,600 miles of 10-ft. navigation, and 70 streams give about 3,200 additional miles of navigation of from 6 to 10 ft. during the greater part of the year, making a total of about 5,800 miles of river navigation 6 ft. and over. These totals, however, do not of course mean that there are any such continuous stretches of inland waterways of these respective depths. The greater number of these streams flow into the Atlantic, but few of the Atlantic rivers have more than 100 miles of navigation over 6 ft. in depth. The longest connected river system is the Mississippi and its principal tributaries, with about 2,500 miles of a depth of 6 ft. and more.

It is evident that rivers of a navigable depth of only 6 ft. can not be used by vessels best suited to ocean or Great Lake navigation, and furthermore, that vessels which give the most economical service on rivers having that depth cannot be used on still shallower rivers. These facts, together with the shortness of the navigable length of most rivers having a 6-ft. depth and their unconnected character, are of great importance in estimating the present value of our inland waterways. It is evident that these waterways constitute by no means a unified system. The predominating importance of through traffic in modern transportation makes this lack of unity a most serious defect, and accounts to a considerable degree for the failure of our waterways to hold their own in competition with the rail system.

CANALS.—About 4,500 miles of canals have been constructed in the United States. Of these, 2,444 miles, costing in all about $80,000,000, have been abandoned.

The height of interest in canal building came about 1827 to 1837, after the success of the Erie Canal. The crisis of 1837 stopped canal building for the time. The introduction of the railroad about this time was also an important factor in this decline. Since then the use of canals has been as a whole steadily decreasing, both in absolute amount and still more markedly in proportion to the total traffic of the country.

This is due in part to improvident original location of the canals, in part to mismanagement and to adverse control of strategic connections and terminals, but above all else to the fundamental inability of the ordinary type of towpath canal to meet modern conditions of transportation. They are thus unable to compete with railroads to any considerable extent.

Notwithstanding these facts, private and state canals still exist. State canals are maintained in New York, Ohio, Illinois, and Louisiana, with a total mileage of nearly 1,360 (including branches and feeders), constructed at a cost estimated at $156,983,538. (See Table II.)

The Ohio and Illinois canals are antiquated waterways, with little traffic; and the Erie Canal, in spite of its important past, is inadequate in its present condition to meet modern traffic demands, although the new barge canal in course of construction by the State of New York is intended to give a revival of this route.

There are 16 private canals in operation in the United States of more than local importance, with an aggregate mileage of 632. (See Table III.)

The capital stock reported by the companies owning these canals is $43,326,539. Their indebtedness as far as reported to the Bureau is $38,307,740. The more important private canals are those comprising the Atlantic coast system, which connect neutral waterways so as to furnish continuous inland communication for light-draft vessels from the sounds of North Carolina to Long Island Sound, though their use for such long-distance traffic is wholly insignificant. Most of these canals have a declining traffic. Little has been done in the way of improving them since the civil war; and several have come into the possession or control of railroads, which appear to discourage their use. The control of canals by private interests does not appear to be successful. Such private control also to some extent impedes free use of the natural waterways with which they are connected.

The "towpath canal," forming a route in itself, and not a mere short connecting link or extension of deep open routes, is wholly outclassed in attempting to meet the demands of modern transportation. Using animal power, they are too slow for most high-grade traffic. The depth of water seldom exceeds 7 ft., so only small boats can be used. Moreover, it is unable to furnish feeder lines, factory sidings, etc., or their equivalents, with the ease of the rail system. The bulk of our traffic is now so largely long-distance that even the longest canal cannot depend on traffic wholly originating and terminating within its limits; and when it thus has also to seek long-distance traffic, for which it is merely one link in the chain, it meets heavy rail competition, difficulties of transshipment, and difficulties of connecting terminals. The struggle for traffic is so keen that the slightest original disadvantage may easily determine the issue.

The question as to the value of a canal for steam-towed barges of 1,000 to 1,500 tons is still to be determined by practical use.

WATERWAY POSSIBILITIES AND LIMITATIONS.—At first glance the possibilities of our present natural system, if properly developed, seem great; but any proper plan of development must first consider in detail a number of physical and economic restrictions, which place very decided limits on waterway traffic, especially inland river traffic. Differences in types of waterways are

TABLE I.—NAMES, NAVIGABLE LENGTHS, AND DEPTHS OF PRINCIPAL RIVERS OF THE UNITED STATES.

River.	Navigable length, miles.	Depth, feet.
Atlantic coast:		
Penobscot	27.0	11.0
Kennebec	44.0	10.0
Connecticut	50.0	10.0
Hudson	153.0	[1]11.0
Delaware	82.0	[2]6.0
Potomac	113.0	18.0
Rappahannock	106.0	7.5
York	41.0	20.0
James	103.8	16.0
Pamlico and Tar	108.0	2.5— 8.5
Cape Fear	145.0	[3]2.0—20.0
Waccamaw	147.0	2.0—12.0
Pedee	172.0	2.0— 8.0
Santee	143.0	4.0
Savannah	250.0	[4]3.0—21.0
St. Johns	276.0	[5]2.0—20.0
Gulf of Mexico:		
Withlacoochee	77.0	2.0— 5.8
Suwanee	135.0	4.0— 5.0
Apalachicola, Flint, & Chattahoochee	465.0	3.0— 6.0
Alabama	312.0	3.0— 5.0
Coosa	198.0	2.0—
Mobile	45.0	14.0
Tombigbee	341.0	3.0— 6.0
Warrior and Black Warrior	154.5	6.0
Pascagoula	55.0	5.0—15.0
Bayou Plaquemine, etc	11.0	.....
Bayou Teche	75.0	4.6
Bayou Lafourche	165.0	3.0
Atchafalaya	156.0	.....
Calcasieu	58.0	3.5—12.0
Sabine	64.0	5.0
Neches	135.0	3.0
Brazos	264.0	4.0
San Jacinto (Buffalo Bayou)	27.0	7.5—18.5
Mississippi system.		
Mississippi	2,429.0	2.0—32.0
Below New Orleans	....	32.0
New Orleans to St. Louis	....	9.0—
St. Louis to St. Paul	....	4.5
Above Minneapolis	....	3.0
Ohio	967.0	4.2—21.0
Monongahela	131.0	7.0
Kanawha	90.0	6.0
Green and Barren	227.0	5.0
Cumberland	518.0	.....
Tennessee	652.0	1.5— 3.0
Missouri	2,414.0	.....
Yazoo	173.0	3.5
Ouachita and Black	436.0	.....
Red (La.-Ark.)	590.0	2.0
Pacific coast:		
San Joaquin	200.0	3.0— 8.0
Sacramento	262.0	2.5— 7.0
Columbia	336.0	[6]3.5—20.0
Willamette	164.0	[7]3.0—22.0
Snake	146.0	3.0— 5.0

[1] 25 feet to Hudson.
[2] 23 feet to Philadelphia.
[3] 20 feet to Wilmington; 8 feet to Kings Bluff, and 2 feet to Fayetteville.
[4] 21 feet to Savannah.
[5] 20 feet to Jacksonville; 5 feet to Sanford.
[6] 20 feet to Vancouver, Wash.; 8 feet to The Dalles.
[7] 20 feet to Portland, Ore.

TABLE II.—STATE CANALS IN OPERATION IN THE UNITED STATES.

Name and location.	Termini.	Length, miles.	Depth, feet.
New York:			
Erie	Buffalo & Albany	355.1	7
Champlain	Troy & Whitehall	81.0	5
Oswego	Oswego & Syracuse	38.0	7
Cayuga & Seneca		25.0	7
Ohio:			
Ohio & Erie	Cleveland & P'tsmouth	326.0	4
Miami & Erie	Toledo & Cincinnati	249.0	4-6
Illinois & Michigan	Chicago & La Salle	102.0	6
Chicago Sanitary & Ship Canal	Chicago & Lockport	33.0	22
New Basin Canal (Louisiana)	New Orleans	6.7	8

TABLE III.—IMPORTANT PRIVATE CANALS IN THE UNITED STATES.

Name of canal.	Termini.	Length, miles.	Depth, feet.
Delaware & Raritan	New Brunswick and Bordentown, N. J.	66.0	8-9
Morris	Philipsburg and Jersey City, N. J.	100.7	5
Lehigh	Coalport & Easton, Pa.	48.0	6
Delaware Division	Easton & Bristol, Pa.	60.0	6
Schuylkill Navigation (Pa.)		90.0	6½
Chesapeake & Delaware (Del. & Md.)		13.6	10
Chesapeake & Ohio	Cumberland, Md., and Georgetown, D. C.	185.0	6-7
Dismal Swamp (Va. and N. C.)		22.1	9
Albemarle & Chesapeake (Va. & N.C.)		11.0	9
Fairfield (N. C.)		4.0	6
Newbern and Beaufort (N. C.)		3.2	5
Old Basin (Louisiana)		2.5	7
Barataria and Lafourche (La.)		7.0	6
Harvey's (La.)		5.35	6
Lake Borgne (La.)		7.0	7
Willamette Falls (Oregon)		.6	6

*Extract from the Report of U. S. Commissioner of Corporations on Transportation by Water in United States.

of vast importance. The great physical differences between deep, open water routes, like the sea or the Lakes, and the shallower, winding, and narrow channels of rivers and canals make the use of the two not at all comparable. Arguments as to one cannot be safely based on the facts of the other. A necessary corresponding difference in types of vessels suitable thereto and now used thereon is a natural consequence. These two conditions impose great restrictions on the complete unification of our system, and what may be called the interchangeability of the floating equipment, and consequently on through shipments.

This lack of unity and of interchangeability is a factor of enormous importance in the use and development of waterways, and their relations with the railroad system.

GOVERNMENT AND STATE IMPROVEMENTS.—Until about 1850 the improvement of navigable streams and the construction of artificial waterways in the United States were principally carried on by the several States and by private corporations. Both classes of enterprises, however, were sometimes encouraged by the United States Government, either by appropriations or, more frequently, by land grants and subscriptions to stock.

Since about 1870, the date of the present form of the rivers and harbors bill, the policy of the United States Government has undergone a radical change. Improvements formerly conducted by States and private corporations have been taken over by the Government, and the amounts appropriated for waterway improvements have been largely increased. The total Federal appropriations for inland river improvements and canals (excluding harbor works) to 1907 have been over $250,000,000.

"Canals owned by the Government form links in through water routes, such as the St. Marys Falls Canal, rather than distinct routes in themselves. The improvement of rivers by the Government has consisted in the removal of obstructions to navigation, the deepening of channels, the construction of revetments, and the development of slack-water navigation by the building of locks and dams for the purpose of maintaining a proper depth. Such improvements have been carried out to some extent on most of the navigable streams of the country, the most comprehensive and connected system of river improvement being that of the Mississippi and some of its important tributaries.

In the development of our waterway system, while undertaken in part by state and later and more extensively by federal power, there has been very little of that cooperation between central and local authority that appears in the best of the European systems. The United States, the States, the counties, levee districts, and municipalities have all spent large sums of money on waterway improvement, but for the most part separately. There has thus been waste, lack of uniformity, lack of comprehensive plan, and especially a lack of any proportionate contribution from localities peculiarly benefited.

While it is true that the entire country is to some extent interested in a good waterway system in each part thereof, nevertheless it must be remembered, as pointed out above, that any complete unification of that system is, for the present at least, physically impossible, and that therefore the traffic on such waterways will remain for a time confined to certain large sections of the country, and the direct benefits thereof will also to a considerable degree be local. European countries have in many cases met a similar condition by a distribution of costs of improvements apportioned upon localities in some ratio to special benefits received.

EQUIPMENT EMPLOYED IN TRANSPORTATION BY WATER.—This includes various classes of craft, such as sailing vessels, steam vessels, gasoline boats, barges and canal boats. Steam vessels show a continuous and rapid increase both in number and tonnage. They have exceeded sailing vessels since 1895 and now have more than double the tonnage. Sailing vessels have declined in importance since 1860. There has also been a notable increase in the number of barges moved by towboats, especially on the Mississippi and its tributaries and on the Atlantic and Gulf coasts, showing an important change in methods of traffic movement. The total number of unrigged craft in the United States in 1889 was 16,937, as compared with 20,263 in 1906.

The total gross tonnage of documented vessels engaged in the domestic trade in 1896 was 3,858,927 tons, and in 1906, 5,735,483 tons, a gain of 1,876,556 tons, or 48%. The tonnage of American vessels engaged in the foreign trade, after reaching its maximum in 1860, has continued to decline except for a slight increase since 1900. In 1896 it was 884,954 tons; and in 1906, 939,486 tons, an increase of only 94,532 tons, or 11%.

The distribution of domestic vessels by districts in 1907 shows that the Atlantic and Gulf coasts are the most important districts, including nearly half of the total documented tonnage and two-thirds of the sailing-vessel tonnage. The Great Lakes have more than one-third of the total documented tonnage and almost half the total documented steam tonnage of the United States.

There has been a notable increase in the size of vessels on the coasts and the Great Lakes, similar to the increase in the size of railroad rolling stock and with similar results in the reduction of transportation costs. As a general rule, and with vessels of the same type, the larger the vessel the lower the cost of carriage.

The size of vessels is, however, limited in nearly all cases by the depth of channels and harbors and by the terminal accommodations. Though the limits vary widely on different classes of waters and at different ports and channels, it may be said in general that a great many vessels have already been built with as great draft as the waterways they use will permit, and that in the case of most of our rivers and canals the maximum draft of vessels is scarcely sufficient to permit very economical operation. This fact has a very important bearing on the possibilities of successful water transportation.

Reference has already been made to differences in depth of different classes of waterways as requiring differences in the size of vessels. These necessitate transshipment in order to secure through transportation, even between points both of which are on waterways. Other differences in physical conditions of different kinds of waterways also prevent the use of the same vessels for through shipments. Thus, vessels used on quiet inland waterways such as smaller inlets and lakes, and on rivers and canals, are neither sufficiently strong nor have they sufficient freeboard to permit them to be used on the Great Lakes or the ocean. Experts even assert that were there enough channel depth to take vessels like the great ore freighters from the Great Lakes to tide water, nevertheless it would be preferable, and almost necessary, to transship at tide water as now. These ore freighters on the Great Lakes are not in model or in structural strength adapted for ocean conditions.

Again, the narrowness and tortuousness of river channels almost entirely prevent the use of sailing vessels, and practically also prevent the use of large steam vessels such as are adapted to the Great Lakes or the ocean. Thus, even on a channel as deep as the Hudson practically no large ocean boats are seen at any distance above New York, though there is 25 ft. of water up to Hudson, 100 miles above New York. Owners do not care to risk the dangers of a restricted channel in large ocean-going craft. Again, in the present state of the art, even light-draft screw steamers cannot be used to advantage on the Mississippi at any considerable distance above New Orleans. Aside from the shallow depth, the need for constant backing and turning at the innumerable bends makes the stern-wheel paddle the only generally successful method. The latter, of course, is useless on the open sea.

Specialization in vessels is not only with respect to the peculiar condition of the waterways, but it has gone further in adapting them to the peculiar conditions of the different kinds of traffic. This adaptation is especially conspicuous in what are known as bulk-cargo vessels; that is, vessels which do not carry package freight, or diversified freight, but which carry one or at most a few kinds of articles in bulk. Thus on the Great Lakes bulk steam vessels for ore or coal, and others for grain, are used to an enormous extent. On the Atlantic and Gulf coasts there are bulk vessels for coal, for oil, for fruit, etc. On the Pacific coast a type of steam schooner has been developed, especially intended for hauling lumber in bulk, and special lumber vessels are also used on the Great Lakes and the Atlantic Ocean. On the Ohio and Mississippi bulk coal barges are extensively used, and other bulk boats to some extent. The use of rafts of logs on the Great Lakes, on the rivers, and on the Pacific coast is a special development of bulk transportation.

This development of the bulk water carrier is highly significant of the peculiar demands of modern transportation. It has been most successful in those cases where the products to be shipped are either on or near the water, and where the destinations to be reached are likewise on or near the water. This is conspicuous, for example, in the case of the coal carried on the Monongahela, Ohio, and Mississippi rivers, most of which is produced at or very near the rivers. At the same time, a large part of the traffic in bulk water carriers reaches those carriers only by rail haul, and a considerable part is delivered by rail to the final destination. This occurs, as on the Lakes, where there is a large uniform supply, a corresponding demand, and a proportionately long water haul connecting the two.

This specialization of types of vessels, as in the various classes of bulk freighters, in the use of gasoline boats on the Mississippi River, etc., indicates the struggle of a system to adapt itself to its conditions. Probably had there been no rail competition, that adaptation would have gone much further and would have approached something like a good operating system, even on the inferior waterways, such as rivers and canals. But in the keen struggle with the railroad for existence, hand-to-mouth methods have been necessary, except on the Great Lakes and the coast, and even there it is to some extent hampered by rail competition. Could the rail and water systems have cooperated instead of competing, it seems probable that a reasonable share of traffic would have produced a modern system of waterway transportation even on our smaller rivers.

From these considerations as to the character of our waterways, and of the vessels used upon them, it is obvious, as already stated, that our inland waterways are far from constituting a unified system, and that, save on the Great Lakes, and in the coal traffic on the Ohio, they are very inferior in efficiency to railroads. In a later part it will be shown that the traffic on rivers and canals of the United States is insignificant in comparison with the rail traffic, or with traffic on the Great Lakes and the coasts. Diversity in channel and navigation conditions, diversity in vessel types, in motive power, and in terminals, as well as in carriers' liability, insurance, freight classification, and traffic contracts generally, have resulted in extreme absence of organization of our water system as a whole. It is thus helpless in competing for long-distance traffic with a modern, unified, standardized system like that of the railroads.

LIMITATIONS OF TRANSSHIPMENT.—It cannot be expected that any reasonable expenditure on our inland waterways will wholly remove these difficulties. It would never pay, except in a few cases, to try so to deepen and improve the rivers and canals as to permit vessels suited to the ocean to use them to any appreciable extent. Transshipment from one water carrier to another will continue to be necessary on most long inland, or partly inland, hauls, even from and to points actually on waterways. Moreover, the greater part of our traffic originates in, or is destined to, points not on waterways, so that transshipment from rail to water will always be necessary on most longer hauls which seek to use the waterways at all.

Since such a large portion of our traffic is long-distance traffic which thus involves transshipment, the question of the future development of our inland waterways resolves itself into one of relative operating expenses and of investments of capital. There is little doubt that for a given distance a continuous water haul in a boat of even moderate size is, and will continue to be, cheaper in operating costs alone than a similar continuous rail haul. If the channels are made deep enough to permit the use of relatively large boats, this greater cheapness will, in some cases, be sufficient to cover also the additional cost of transshipment. The question is, however, whether besides doing so it will afford enough saving to cover reasonable interest on the investment in waterways and water terminals necessary to secure such measure of efficiency. This is a question which can only be answered after most exhaustive engineering estimates of the cost of constructing waterways of different depths and most thorough investigations of operating expenses of different kinds and sizes of vessels, in comparison with expenses of rail transportation.

It should be noted, however, in considering how far expenditures of capital for improved water routes are advisable, that in some cases the railroads have already reached the limit of their carrying capacity and that they will have to make additional investments of capital if they are to meet the demands of transportation. The relative amount of investment necessary thus to increase the facilities of the railroads, as compared with the investment necessary to afford adequate water routes and terminals, must therefore likewise be taken into account.

BUSINESS ORGANIZATION AND FINANCES. OWNERSHIP.—Prior to the development of steam navigation vessels were usually owned and operated by individuals or small groups of individuals. Sailing vessels are even now more commonly thus owned than by corporations. The corporate form, however, has gained steadily, at first with steam vessels and later with sailing vessels and barge lines. There was a total of 37,321 vessels, with a total gross tonnage of 12,893,429 tons, and valued at $507,973,121, included in the census of 1906. Of these, corporations owned more than one-half of the number, and more than three-fourths of the total tonnage and value. Of steam vessels corporations controlled 42.6%, representing 87.6% of the gross steam tonnage; and of freight and passenger vessels running on regular lines they controlled 56.8%, with 91% of that tonnage. The proportion of vessel tonnage owned by corporations is everywhere larger than the proportion of the number of vessels so owned, indicating that the vessels owned by corporations are usually the larger ones.

In 1906 the average size of vessels owned by individuals was 113 tons, of vessels owned by firms 223 tons, and of vessels owned by corporations 526 tons. Many steamship corporations, however, are essentially private affairs with few stockholders. They often are controlled by a few individuals, often of the same family. Navigation companies have been in most cases conservatively capitalized and their stock not offered in the open market. But in recent years there have been some notable changes in these respects. By expansion or consolidation there have been developed quite a number of great corporations with large fleets and operating many lines over a wide territory. The most striking instance of consolidation has been that of the Consolidated Steamship Lines, commonly known as the Morse Combination, including half a dozen important Atlantic coast lines, covering a large share of the regular packet-line business in this important district.

An especial feature has been the recent acquisition and operation of steamer and barge lines by railroad companies and by industrial and trading concerns, including several of the great combinations, such as the United

States Steel Corporation, the Standard Oil Co., the Southern Pacific (Railroad) Co., the New York, New Haven & Hartford Railroad Co., the New York Central Railroad, the Pennsylvania Railroad, and others.

FINANCIAL.—The list of concerns set forth in the tables in this report is by no means complete, while the information is also inadequate for a satisfactory report on their financial operations. The scope of the inquiry was limited to a few of the most important facts relating to the capital stock (in case of corporations), indebtedness, gross earnings, and operating expenses. But even on these points the failure of many concerns to make any report renders the details incomplete, and only a few general and tentative conclusions can be drawn therefrom.

GOVERNMENT REGULATION.—The interstate-commerce acts have been applied only in a limited degree to carriers by water; and it is uncertain how far the provisions of these acts as to rates and carriers' liability are intended to apply to water lines. This is in marked contrast with the close supervision over vessels and their officers for the protection of life. In this latter respect water traffic has long been far more closely regulated by government control than railroad traffic. The detailed strictness with which the construction, operation, and maintenance of vessels is watched by the Government, and its complete system of inspection thereof, is in very marked contrast to the almost complete absence of such control over railroads. This difference, also, has borne with some disproportionate weight on the water traffic.

On the other hand, governmental control of rates has been much more complete over railways than over waterways.

The relation between water and rail routes and rates will be more fully considered in a subsequent part of this report.

MARINE INSURANCE.—Marine insurance charges are an important factor as an addition to the cost of transportation by water, and they must be taken into consideration in comparing the relative economy of rail and water routes.

Since the floating equipment and cargoes protected by insurance are of various kinds, there is a similar diversity in the types and stipulations of insurance policies. As regards vessels, there are river hull policies, ferryboat hull policies, inland hull, tug, yacht, schooner, and steamboat policies. As regards cargoes, there are cotton policies, coal, lake-cargo, and river-cargo policies. There are also blanket policies on hull and cargo and on all the vessels and cargo of particular lines.

There is considerable variety both in the manner of effecting insurance on vessels and cargoes and in rates of premium, not only on the different waterways of the country, but also in respect to the several types of vessels and the freight transported. Inland policies are peculiar in the restrictions upon the routes on which the craft may operate, and in the varying rates of premium on different streams. Some vessels carry only fire insurance, others both fire and marine insurance, and others fire, marine, and port insurance. Often no vessel insurance or insurance of any kind is carried. Some vessel owners are their own insurers, providing special reserve funds for that purpose. Insurance on vessels is usually by the year, often with lay-up clauses applicable to the period during which navigation is suspended.

Cargo insurance in some cases is entirely attended to by the vessel owner on behalf of the shipper or consignee. Frequently, however, it is effected on each shipment by the shipper and continues until the vessel reaches the port of destination or for a certain time thereafter. In other cases, however, the vessel owner or owner of cargo lots takes out a blanket policy covering all shipments for a given period.

Most river steamboats and package freight shipments seem to be protected by insurance. Differences in the conditions of river improvement, navigability, and river dangers are reflected in the insurance rates. In fact, the cost of insurance is often a determining factor as between rail and river shipments. The rail carrier being practically, by law, the insurer of the freight, insurance on rail freight is a negligible consideration to the shipper. But on water, and especially river traffic, this is by no means so. Selected river insurance rates have been as follows:

A company operating on Virginia and Maryland waterways reports that the insurance premium for a recent fiscal year amounted to 3.1% of the gross receipts. Cargo insurance on the Red River is now 0.9% of the value of the goods. On the Mississippi River from Vicksburg to New Orleans cargo insurance is 45 cts. per $100, or about 20 cts. per bale of cotton, which must be added to the freight rate of 75 cts. per bale, thus making the insurance nearly one-third of the freight.

These high and varying rates of insurance not only increase the cost of transportation, but they also indicate the dangers of such traffic and add to the uncertainty and lack of uniformity so detrimental to shippers' interests.

There seems to be a general tendency for marine insurers to work in groups or combinations. A large proportion of the total risks assumed in the United States are taken by foreign companies. In the United States, marine insurance is largely written by corporations instead of by such associations as Lloyds, in England. There are nine important domestic companies, of which only one devotes itself exclusively to marine indemnity, the others combining fire and marine business, their fire-insurance business being larger than the marine.

Taxation of Vessels and of Navigation Companies.

The power over interstate and foreign commerce vested in Congress, together with the specific prohibitions of the Constitution against state tonnage and customs duties without the consent of Congress, restricts the taxing power of the States over navigation as such, while leaving the States free to tax vessels like other property within their respective jurisdictions. Thus a tax on interstate freight or passenger traffic as such is unconstitutional, and the privilege of navigating public waters is not subject to state license tax; but States may under some conditions impose franchise taxes upon the gross receipts of transportation companies, even though part of them be derived from interstate traffic, or upon the capital stock of companies engaged in interstate commerce.

Tonnage taxes, no matter in what form, may not be imposed by a State on vessels simply because of their use of or presence on a public waterway, although where a State has made improvements on a waterway it may charge a toll for the use of those improvements in reasonable proportion to the cost and maintenance of such improvements, even if such toll be calculated on tonnage.

Furthermore, a State or municipality may charge wharfage fees for the use of landing places provided by it, even if such fees be fixed on the basis of tonnage.

STATE TAXATION.—One of the common methods employed in the taxation of navigation companies appears to be an assessment of the capital stock at its market value, after deducting therefrom the assessed valuation of the corporation's vessels and other tangible property which are taxed like similar property belonging to individuals. The States using this method are Massachusetts, North Carolina, Alabama, Kentucky, Illinois, Kansas, Minnesota, and North and South Dakota. Several States, however, impose relatively light franchise or license taxes measured by the par value of the stock, and without regard to any taxes assessed upon the tangible property. These States are New Jersey, New York, Maine, Delaware, Ohio, West Virginia, Georgia, Texas, Colorado, and California. A number of others tax gross or net receipts as well as the capital stock and tangible property. These States are New York, Pennsylvania, Maryland, Virginia, and South Carolina.

In most of the States water craft of all kinds are assessed and taxed like other personal property, either at the residence of the owners or, what is often the same thing, at the port of enrollment, registry, or license. A few States provide for taxes measured by the gross or net tonnage in lieu of other taxes on such property, on the assumption that the value of vessels is roughly in proportion to size. Vessels engaged in the foreign trade are in several States favored either with exemption from taxation or with very light taxes based upon their net earnings. Massachusetts, which formerly taxed the net earnings or dividends in such cases, now assess the property, whether belonging to individuals or corporations, at the low rate of one-third of 1%—about one-fifth of the rate of taxation on other property. A low rate of taxation on vessel property is probably an advantage to the State imposing it, because of the ease with which taxation can be evaded in respect of property so easily movable.

FEDERAL TAX.—There is no Federal tax on American vessels in domestic trade. The National Government imposes a tonnage tax on vessels entered at ports of the United States from foreign ports, at the rate of 3 cts. per ton in the case of vessels from near-by foreign ports and 6 cts. per ton when from more distant places, but not to exceed 15 cts. per ton per annum in the former case or 30 cts. per ton per annum in the latter case.

THE COLLAPSE OF A SCAFFOLDING in the shaft of the Kamphausen coal pit, Saarbrucken, Germany, Aug. 31, resulted in the death of eight men. The men were at work on the scaffold and were dropped several hundred feet to the bottom of the shaft.

AN EXPLOSION OF DYNAMITE near Tyrone, Pa., Sept. 6, wrecked all the buildings at the quarry of the American Lime & Stone Co., together with the residence of one of the company's officials. Reports state that one ton of dynamite stored at the quarry was exploded. The explosion is said to have been brought about intentionally by discharged quarry workmen.

WRECKERS DERAILED AN EXPRESS TRAIN on the Baltimore & Ohio R. R. Sept. 4 near New Castle Junction, Pa. The train, No. 5, known as the "Royal Blue" flyer, consisted of two Pullman cars, two day coaches and a baggage car, all of which were overturned. Three persons were killed and 17 others seriously injured. An examination of the track after the accident showed that a rail joint had been removed, a number of spikes pulled and the rail ends deflected in such a way as to form a derailing switch.

A RAILWAY ACCIDENT Sept. 3 at the western end of the Radebaugh tunnel on the Pennsylvania R. R. near Greensburg, Pa., resulted in the death of four track laborers. A train issuing from the tunnel at full speed was not noticed by the men until too late to escape it on account of the noise of an eastbound train just entering the tunnel.

A RAILWAY TRAIN STRUCK A STREET CAR Sept. 2 at Des Moines, Iowa, on a Chicago, Rock Island & Pacific Ry. grade crossing. Thirteen of the passengers in the crowded car were severely injured. The car was picked up on the locomotive pilot, carried a distance of 50 ft. and jammed against the end-posts of the steel railway bridge across the Des Moines River. On account of the precarious position of the car, which was in danger of falling into the water 20 ft. below, there was considerable difficulty in removing the injured passengers.

A RAILWAY TRAIN STRUCK AN AUTOMOBILE Aug. 29 on a grade crossing of the Chicago, Rock Island & Pacific Ry. near Vigus, Mo. The five occupants of the automobile, two men, two women and a three-year-old child, lost their lives. High brush on either side of the road at the point where the accident occurred prevented a clear view of the railway track and likewise concealed the approaching automobile from the engine driver. The train, which was running at about 35 mi. per hr., struck the automobile squarely in the center so that it was picked up on the locomotive pilot and carried a quarter of a mile down the track to the point where the train was stopped. The two men, who were on the front seat, were thrown out at the moment of collision and landed in a ditch 15 ft. from the tracks. One of them was found dead, face-down in the shallow water of the ditch and the other died in a hospital a few hours later. The two women were wedged on the pilot with the wreckage of the automobile, while the child was carried a distance of about 150 yds. from the crossing before being tossed to one side.

An automobile was struck by a train on the Pennsylvania R. R. Sept. 2 at Douglassville, Pa. The four persons in the automobile were thrown out and three of them were killed outright while the fourth was probably fatally injured. The gasoline tank burst and the resulting fire consumed the wreckage of the automobile.

A train struck an automobile Sept. 5 at Bay City, Mich., at a Michigan Central R. R. grade crossing. Three persons were killed and two seriously injured. The chauffeur, who survived, stated that although approaching the crossing slowly he was not aware of the train until it was close upon him. In his excitement he allowed the engine to stop and the automobile came to rest directly on the tracks. At this crossing the road crosses the track at a sharp angle and the train and automobile were going in nearly the same direction.

FOREST FIRES IN BRITISH COLUMBIA are again burning in the vicinity of Fernie, which was completely wiped out in the fires of last year. A number of separate fires are burning throughout the southeastern portion of the province. The town of Roseberry has been destroyed, together with a large amount of standing timber, and many bridges and buildings of the Great Northern Ry. and Canadian Pacific Ry. have been consumed.

FIRE DESTROYED AN ORPHANAGE at Rockaway Park, Long Island, Aug. 30. Although there were 500 children between the ages of 3 and 15 years and 100 babies under two years old in the building when the fire broke out, only seven lost their lives. The building was known as St. Malachy's Roman Catholic Orphan Asylum and was a three-story wooden structure with a boiler-room and laundry in the basement of the east wing. The fire started in the laundry and was first discovered at about eight o'clock in the evening, shortly after the children had been put to bed. A heavy loss of life was prevented by the calm behavior of the Catholic Sisters in charge and the discipline developed by previous "fire drills."

A TUNNEL FIRE on the new Billings & Northern branch of the Great Northern Ry. at Great Falls, Mont., destroyed a large amount of timber lining. The fire was extinguished after several days' work. It is stated that resumption of traffic may not have to wait for rebuilding of the lining, as the main part of the tunnel is in firm rock.

THE MEXICAN FLOODS were even more serious than we stated last week, according to later reports. Damage was extremely wide-spread in northeastern Mexico, and the number of dead is now placed at about 3,000. The railroads alone claim a money loss of two to four

million dollars after only partial survey of the damage. There are reports that a tidal wave destroyed Soto la Marina. A new rainstorm at Tula on Sept. 5 caused a flood which increased the damage there materially. The lower Rio Grande reports very high water and considerable destruction; the town of Camargo, Mex., is said to be practically destroyed. This flood is due directly to the flood in the Santa Catarina River, which inundated Monterey. The loss of life and property destruction at Monterey is the largest item in the flood ruin. Appeals for aid for Monterey have been sent out generally to Mexico and the United States.

THE "LUSITANIA" Cunard transatlantic steamship, reduced the record for passage between Daunt's Rock and the Sandy Hook lightship to 4 days, 11 hrs., 42 min., on the trip ending at New York Sept. 2. The best previous record over this course, 4 days, 14 hrs., 38 min., was recently established by the sister ship, "Mauretania." However, an equivalent record of 4 days, 13 hrs., 24 min. was made by the "Mauretania" Aug. 30 over a course reported slightly different. The "Lusitania" is, by this trip, the first westbound steamer to land its passengers within five days from the sailing port.

THE NORTH POLE was reached by Dr. F. A. Cook, of Brooklyn, N. Y., on April 21, 1908, according to despatches received from him on Sept. 1, when he reached the Shetland Islands on his way south. He reports starting from Annatok on the northwestern coast of Greenland, Feb. 19, 1908, with ten other men, 103 dogs and eleven sledges. Smith Sound and Ellsmereland were crossed, and the route was laid up Nansen Sound to Lands End of Grantland. The last point of land was left March 18 with five men forming the party. March 15 the party was reduced to Dr. Cook and two natives with 26 dogs for the dash of 460 miles. On March 30 when at latitude 81° 47', longitude 86° 36' land was seen to the west but it was not explored. On April 14, latitude 88° 21', longitude 95° 52' was attained, and on April 21 latitude 90° was reached on ice with no land in sight. The return was started April 23 and at first long distances were easily made. Below the latitude of 87°, however, troubles began to multiply. On May 24 a point at latitude 84°, longitude 97° was reached on broken ice-fields. From this time to July, attempts were made to cross eastward from Crown Prince Gustav Sea but on account of a westward drift of the ice fields the party was forced south to Jones Sound far to the southwest of the starting place in Greenland. The winter of 1908 was spent at Cape Sparbo, where the party had to rely on scarce game to furnish food. On Feb. 18, 1909, the start was made for Annatok and Upernavik, which was reached May 21, 1909.

On Sept. 6, Commander Robert E. Peary, U. S. N., reported from Cape Ray, N. F., that his expedition reached the North Pole on April 6, 1909. Both Cook and Peary are on their way to the United States and full details of their journeys will soon be available.

A CLOUDBURST AT RAWHIDE, NEV., Aug. 31, is reported to have destroyed more than 150 buildings, rendering 500 persons homeless. Reports state that the streets of the mining camp were swept by a flood of water 12 ft. deep.

A FRENCH AVIATOR, E. Lefebvre, was killed at Juvisy-sur-Orge, France, on Sept. 7, while practicing with a Wright flyer which he was testing. He had been a contestant at the recent aerial meet at Rheims where he became noted for his daring and reckless performances.

Personals.

Mr. R. J. Lockwood, Resident Engineer of the Marshall & East Texas Ry., has been promoted to be Chief Engineer.

Mr. A. H. Sabin, a consulting chemical engineer of Flushing, N. Y., has been elected a fellow and member of the honorary council of the North British Academy of Arts and Sciences.

Mr. W. L. Kellogg, Master Mechanic of the Pere Marquette R. R., has been appointed Superintendent of Motive Power of the Cincinnati, Hamilton & Dayton Ry., with headquarters at Lima, Ohio.

Mr. H. S. Dickey, General Superintendent of the Winona Interurban Ry., with headquarters at Warsaw, Ind., has resigned and is succeeded by Mr. C. O. Sullivan, formerly Assistant General Superintendent.

Mr. Harry H. Stock, M. Am. Inst. M. E., Editor of "Mines and Minerals," Scranton, Pa., has been selected, according to reports, to be the head of the new department of mining engineering of the University of Illinois.

Mr. Harold K. Weld, Assoc. Am. Inst. E. E., has resigned his position in the engineering department of the Chicago Telephone Co., and has entered the sales department of the McRoy Clay Works, Chicago, manufacturers of underground conduits.

Mr. W. H. Caton and Mr. H. S. Hackbusch have formed a partnership as civil and concrete engineers under the firm name of Caton & Hackbusch, with offices at Muskogee, Okla. The new partnership will continue the practice formerly conducted by Mr. Hackbusch alone.

Mr. J. H. Philips, M. Am. Soc. C. E., has become Principal Assistant to the Engineer and Superintendent of the Essex Co. Park Commission, Branch Brook Park, Newark, N. J. In our issue of Sept. 2, Mr. Philips' new title was erroneously given as chief engineer of the park commission.

Mr. M. M. Leydendecker has been appointed General Superintendent of the Texas-Mexican Ry. to succeed Mr. S. W. De Wolf who has resigned to take charge of private property interests of Mr. E. N. Brown, President of the National Rys. of Mexico. Mr. Leydendecker was formerly Chief Dispatcher at Laredo, Tex.

Mr. J. A. S. Redfield, recently Division Engineer of the Northern Iowa and Sioux City divisions of the Chicago & North Western Ry., has been appointed Resident Engineer of the Sioux City, Dakota & North Western Ry., with headquarters at Hawarden, Iowa. Mr. R. W. Richardson has been appointed Acting Division Engineer to succeed Mr. Redfield.

Mr. Philip Burgess, Assoc. M. Am. Soc. C. E.; Mr. A. E. Kimberly and Mr. Chas. F. Long have severed their connection with the Ohio State Board of Health and have opened offices in the Columbus Savings & Trust Bldg., Columbus, Ohio, for the practice of consulting sanitary and chemical engineering under the firm name of Burgess, Kimberly & Long.

Mr. C. A. Wilson, M. Am. Soc. C. E., Chief Engineer of the Cincinnati, Hamilton & Dayton Ry., has resigned that position but is still connected with the railway company in an advisory capacity. Mr. F. H. Alfred, M. Am. Soc. C. E., whose appointment as Assistant to the President of the C., H. & D. Ry. was noted in our issue of Sept. 2, has been placed in charge of the engineering department.

Mr. G. H. Groce, Superintendent of Telegraph and Signals of the Illinois Central Ry., has been appointed Assistant to the General Manager, and the former position has been abolished. Mr. N. E. Baker is Signal Engineer. Mr. F. T. Wilbur (at Chicago) has been appointed Superintendent of Telegraph for lines north of the Ohio River, and Mr. B. Weeks (at Memphis, Tenn.) for the lines south of that point.

Mr. Schuyler Hazard, M. Am. Soc. C. E., formerly Division Engineer of the New York Central & Hudson River R. R. and later Roadway Engineer of the Southern Ry. Co., has resigned as Resident Engineer of the New York Contracting Co., New York City, to become General Manager and Chief Engineer of the Orleans County Quarry Co., producers of Medina stone, in charge of production, sales and engineering, with headquarters at Albion, N. Y.

Obituary.

William H. Singer, M. Am. Inst. M. E., of Pittsburg, Pa., died Sept. 4 at the New Watch Hill House, Watch Hill, R. I., where he had been spending the summer with his family. His death was the result of injuries received when his automobile was recently overturned in a ditch and its occupants thrown out. Mr. Singer was the founder of Singer, Nimick & Co., a pioneer steel concern, and accumulated a fortune estimated at $20,000,000 in the iron and steel business.

William Ford Stanley, founder of W. F. Stanley & Co., Ltd., London, Eng., makers of surveying and drawing instruments, died Aug. 14 at the age of 81 years. Mr. Stanley was born at Buntington, Hertfordshire, Eng., and was apprenticed at an early age to his father, a mechanical engineer. Beginning business for himself in 1854 as a metal and ivory worker, he later turned his attention to the design and manufacture of scientific instruments for the British Government and founded the firm of which he was for 50 years the head. He conducted a private research into the properties and motion of fluids and was the author of a book on this subject and of text-books on surveying and drawing instruments.

Charles F. Wingate, of New York City, a sanitary engineer, died Aug. 31 at his summer home in Twilight Park, Greene Co., N. Y. Mr. Wingate was born in New York City in 1847 and was educated in the New York public schools and at Cooper Institute. He worked at first as a ship broker and afterward was engaged for twelve years in newspaper work. He was for some time the New York correspondent of the Springfield, Mass., "Republican." Later he took up sanitary engineering and was the author of several articles on problems of sanitation. In 1887 he founded Twilight Park in the Catskill Mts., which he has since made his summer home and which has become a fashionable resort. Most of Mr. Wingate's life had been spent in Brooklyn, but for the past four years he had been living in Manhattan.

Henry W. Brinckerhoff, M. Am. Soc. C. E., of New York City, died Sept. 7 at his home in Bay Ridge, Brooklyn, N. Y., aged 65 years. Mr. Brinckerhoff was born at Ossining, N. Y., and was educated at Harvard University, of which he was a graduate in the class of 1866, and at the Lawrence Scientific School. He served for a time as a civil engineer at the New York Navy Yard and was later engaged in engineering work in connection with the Jersey City water-works, West Shore R. R., Sixth Ave. elevated railway in New York City, the Brooklyn Bridge, New York Steam Co., and the Tehuantepec Ship Ry. He was at one time Managing Editor of the "Engineering Record" and was Office Engineer of the Broadway Cable Road, New York City. In 1889 Mr. Brinckerhoff made the first report on the failure of the South Fork Dam, which caused the destruction of Johnstown, Pa. During recent years he has maintained an office in New York City as a consulting engineer.

Engineering Societies.

COMING MEETINGS.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Sept. 14-16. Annual convention at Atlantic City, N. J. Secy., Frank P. Foster, Corning, N. Y.

MICHIGAN GAS ASSOCIATION.
Sept. 14-17. Annual meeting on board steamer sailing from Detroit, Mich. Secy., A. P. Ewing, Gas Office Bldg., Detroit, Mich.

ROADMASTERS' AND MAINTENANCE OF WAY ASSOCIATION.
Sept. 14-17. Annual convention at Washington, D. C. Secy., Walter E. Emery, Peoria & Pekin Union Ry., Peoria, Ill.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 14-17. Annual convention at Niagara Falls, N. Y. Secy., A. P. Dane, Boston & Maine R. R., Reading, Mass.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Sept. 16-Oct. 12. Annual convention and excursion on special train starting at Chicago, Ill. Secy., R. W. Raymond, 29 W. 39th St., New York City.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 25-27. Annual convention at New York City. Secy., P. S. Millar, 25 W. 39th St., New York City.

AMERICAN MINING CONGRESS.
Sept. 27-Oct. 2. Annual session at Goldfield, Nev. Secy., J. F. Callbreath, Jr., Denver, Colo.

AMERICAN STREET & INTERURBAN RAILWAY ASSOCIATION.
Oct. 4-8. Annual convention at Denver, Colo. Secy., Bernard V. Swenson, 29 W. 39th St., New York City.

RAILWAY SIGNAL ASSOCIATION.
Oct. 12-14. Annual convention at Louisville, Ky. Secy., C. C. Rosenberg, Bethlehem, Pa.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 19. Annual convention at Jacksonville, Fla. Secy., S. F. Patterson, Boston & Maine R. R., Concord, N. H.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Oct. 19-22. Annual convention at Richmond, Va. Secy., Chas. O. Probst, Columbus, Ohio.

AMERICAN GAS INSTITUTE.
Oct. 20. Annual meeting at Detroit, Mich. Secy., A. B. Beadle, 29 W. 39th St., New York City.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Nov. 9-11. Annual convention at Little Rock, Ark. Secy., A. P. Folwell, 512 Flatiron Bldg., New York City.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Nov. 16. Annual meeting at Washington, D. C. Secy., Martin S. Decker, Albany, N. Y.

VERMONT ELECTRICAL ASSOCIATION.—The eighth annual meeting will be held at Burlington, Vt., Sept. 15-16. The second day of the meeting will be devoted to a trip to Bluff Point on Lake Champlain. The secretary is Mr. A. B. Marsden, Manchester Center, Vt.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.—At the 14th annual convention, Sept. 14-16 at Atlantic City, N. J., papers will be presented on the following subjects: "The Construction of Underground Conduits," "Grounding of Alternating Current Secondaries," "National Electric Code and Its Proper Interpretation," "Locating Faults in Fire and Police Telegraph Circuits Without Opening the Lines," "Progress of the Fire Alarm Telegraph," "The Storage Battery for Fire and Police Telegraph Systems."

ASSOCIATION OF EDISON ILLUMINATING COMPANIES.—The 30th annual convention was held at Briarcliff Lodge, Briarcliff Manor, N. Y., Aug. 30-Sept. 2. Among the papers presented were the following: "Special Illumination Features of the Hudson-Fulton Celebration," "The Tungsten Lamp Situation in Various Cities," "Prepayment Meters," "The Central Station and the Electric Vehicle." Mr. Thomas E. Murray, M. Am. Inst. E. E., Second Vice-President of the New York Edison Co., was elected President of the association for the coming year.

NATIONAL ASSOCIATION OF COTTON MANUFACTURERS.—Semi-annual meeting No. 87 will be held at Bretton Woods, N. H., Sept. 22-23. Among the subjects of the papers to be presented are the following: "Accidents to Cotton-Mill Operatives," "Commercial Values of Coal," "Cork Inserts as Applied to Textile Machinery," "Housing Mill Employees," "Legal Relation Existing Between Mill and Selling Agent," "Pneumatic Service for Cleaning Textile Machinery," "Sanitary Considerations in Ventilating and Humidifying Cotton Mills," "Reinforced Concrete for Mill Construction."